VOL. LIII. AUGUST, 1927. No. 6.

REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

This Society is not responsible for any statement made or opinion expressed in its publications.

CONTENTS

Papers: PAGE

The Panama Canal: Address at the Annual Convention in Denver, Colo., July 13, 1927.
By JOHN F. STEVENS, PRESIDENT, AM. SOC. C. E. 1047

Tides and Their Engineering Aspects.
By G. T. RUDE, M. AM. SOC. C. E. 1069

Notes on Arched Gravity Dams.
By B. F. JAKOBSEN, M. AM. SOC. C. E. 1135

Preparing the Groundwork for a City: The Regrading of Seattle, Washington.
By ARTHUR H. DIMOCK, M. AM. SOC. C. E. 1143

Re-Arrangement of a Business District: Changes in Recent Years in Pittsburgh, Pennsylvania.
By NATHAN SCHEIN, M. AM. SOC. C. E. 1161

The Planning of the Industrial City of Longview, Washington.
By S. HERBERT HARE, ESQ. 1177

Modern Crushed Stone and Gravel Surfacing.
By J. W. HOOVER, ESQ. 1185

Relation of Road Type to Tire Wear.
By O. L. WALLER and H. E. PHELPS, MEMBERS, AM. SOC. C. E. 1189

Stream Pollution in the Pacific Northwest.
By WILLIAM F. ALLISON, M. AM. SOC. C. E. 1207

The Sewage Disposal Problem of Los Angeles, California.
By WILLIS T. KNOWLTON, M. AM. SOC. C. E. 1213

The Work of the Los Angeles County (California) Sanitation Districts.
By ALBERT K. WARREN, ASSOC. M. AM. SOC. C. E. 1221

The Lake Washington Ship Canal, Washington.
By W. J. BARDEN and A. W. SARGENT, MEMBERS, AM. SOC. C. E. 1227

The Work of the Port of Seattle, Washington.
By GEORGE F. COTTERILL, ESQ. 1257

Discussions:

Automobile Hazard in Cities and Its Reduction.
By MESSRS. WILLIAM T. LYLE, M. O. ELDRIDGE, C. W. STARK, L. G. HOLLERAN, S. JOHANNESSON, W. GRAHAM COLE, HAROLD M. LEWIS, J. K. FINCH, G. G. KELCEY, RAYMOND A. O'HARA, JOHN A. MILLER, JR., and T. KENNARD THOMSON. 1269

Water Supply for Army Railways in France.
By WILLIAM G. ATWOOD, M. AM. SOC. C. E. 1293

Probability of Flood Flows.
By MESSRS. H. ALDEN FOSTER, W. P. CREAGER, A. STREIFF, and SIGURD ELIASSEN 1297

The Eye-Bar Cable Suspension Bridge at Florianopolis, Brazil.
By MESSRS. J. A. L. WADDELL, SPENCER MILLER, LLOYD G. FROST, C. G. EMIL LARSSON, and CLYDE T. MORRIS 1307

Civil Engineering Features of the Hell Gate Station.
By MESSRS. GEORGE A. ORROK and A. STREIFF 1317

The Relation of Highway Transportation to the Railway.
By MESSRS. JOHN V. HANNA, E. A. HADLEY, and F. G. JONAH 1319

Urban and Interurban Buses.
By MESSRS. ROBERT P. WOODS, FRED G. BUFFE, B. H. PIEPMEIER, C. E. SMITH, WINTERS HAYDOCK, JOHN W. REID, and LUCIUS S. STORRS 1329

The Logging and Lumbering Industries of the Pacific Northwest: A Symposium.
By MESSRS. R. H. STOCK, HUGO WINKENWERDER, and C. S. CHAPMAN 1345

Some Phases of Irrigation Financing.
By MESSRS. M. C. HINDERLIDER, C. E. GRUNSKY, JOSEPH JACOBS, and H. F. DUNHAM 1351

State Reclamation in Washington.
By MESSRS. J. RHEA LUPER, W. G. SWENDSEN, J. C. STEVENS, O. L. WALLER, and JOSEPH JACOBS 1357

Memoirs:

DESMOND FITZGERALD, PAST-PRESIDENT, AM. SOC. C. E. 1365
CORNELIUS CONWAY FELTON BENT, M. AM. SOC. C. E. 1370
WILLIAM HENRY FINLEY, M. AM. SOC. C. E. 1372
GEORGE ALEC HARWOOD, M. AM. SOC. C. E. 1374
CHARLES ALBERT JUDSON, M. AM. SOC. C. E. 1376
JOHN CURTIS PATTERSON, M. AM. SOC. C. E. 1378
HORACE AUGUSTUS SUMNER, M. AM. SOC. C. E. 1379
PETRUS WAHLMAN, M. AM. SOC. C. E. 1381
PHILIP ALBERT WELKER, M. AM. SOC. C. E. 1382
OSCAR ERNEST BULKELEY, ASSOC. M. AM. SOC. C. E. 1384
HENRY GARNER HARPER, ASSOC. M. AM. SOC. C. E. 1386
HOWARD B. MERRICK, ASSOC. M. AM. SOC. C. E. 1387
ROBERT BRUCE ROBINSON, ASSOC. M. AM. SOC. C. E. 1390
CLARENCE HARD THOMPSON, ASSOC. M. AM. SOC. C. E. 1392

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE PANAMA CANAL

ADDRESS AT THE ANNUAL CONVENTION IN DENVER, COLO., JULY 13, 1927

By John F. Stevens,* President, Am. Soc. C. E.

The epic story of Jason and his fellow Argonauts, which describes their quest of the Golden Fleece, contains nothing more of romance than does the recital of the adventures of those bold spirits whose daring, skill, and hardihood, so ably carried on the work of Columbus in giving to the world the continents of the Western Hemisphere.

Columbus, skirting the west shore of the Caribbean Sea in his futile search for a short passage to Cathay with its fabled riches of gold, silver, and precious stones, visited the shores of Panama on his fourth and last voyage in 1502. He named the harbor of Porto Bello and examined Limon Bay, which bay is now the Atlantic entrance to the Panama Canal.

All history points to the conclusion that Columbus died in the belief that the lands which he had discovered were a part of the continent of Asia. The cosmography of those far away days was incomplete and misleading, but it was shortly to be extended, modified, and corrected by a group of adventurers, the story of whose exploits would furnish material for another immortal Odyssey.

Magellan, the great navigator, discovered the straits which bear his name in 1520, and at that time he named the vast body of water to the westward the Pacific Ocean, by reason of its calm and peaceful aspect as he saw it. Sir Francis Drake, probably England's most picturesque and most unscrupulous adventurer of all time, with the exception of Henry Morgan, visited the Isthmus in 1570, captured the Spanish town of Nombre de Dios and burned Porto Bello. Crossing the Isthmus to a high point he saw the waters of the great ocean to the westward. Fifty-seven years before Drake, however, Balboa had crossed the Isthmus, somewhat southeast of the line of the present canal,

* New York, N. Y.

and undoubtedly was the first white man to view the Pacific Ocean. Balboa later fell a victim to the jealousy of the Governor of Panama and was beheaded by him.

The hope of finding a hidden strait connecting the two great oceans, persisted long years after Balboa's discovery, and it is a most interesting fact of history, that it was four hundred years, almost to a day, September 26, 1913, when such a strait was given to the world, but it was not a strait made by Nature, but by man—the Panama Canal.

Sir Francis Drake circumnavigated the globe in 1578 and 1579, probably touched our western coast near the Golden Gate, and returned to England by way of the Cape of Good Hope, the southern point of Africa. Following him were other historical navigators and the result of all their voyages confirmed as a fact, that only a narrow neck of land, fifty miles in width, separated the two oceans at what is now known as the Isthmus of Panama. Morgan, in 1671, crossed the Isthmus with a small force and after enduring terrific hardships captured, sacked, and burned the Spanish town of Panama, and butchered many of its inhabitants. The remainder after living in the jungles for a year or more, built the nucleus of what is now the present City of Panama, a city of about 50 000 or 60 000 people. Morgan used his ill-gotten wealth so wisely that he became Sir Henry and died in his bed an English Christian Knight. The Spaniards, with cross in one hand and sword in the other, proclaimed that their purpose was to carry their religious faith to heretic people, but they were far more actuated in their operations by their thirst for the riches supposed to exist in those unknown lands. By enforced native labor they built a stone-paved pack trail about 2 m. in width across the Isthmus from the Atlantic to the Gulf of Panama, over which was carried the tons of silver which Pizarro and his fellow bandits, the conquerors of Peru, wrested from the hapless people of that unfortunate country. Remnants of this old trail still existed in 1905 and it was my pleasure to ride over some of them in that year. The Panama Railroad was not built until about 1852 and then as the result of the rush of travel consequent upon the discovery of gold in California in 1849.

As nothing in the nature of a strait could be found north of Magellan, which would allow of the passage of ships from ocean to ocean, the persistent leaders began to talk of making one. Saavedra, who accompanied Balboa on the latter's visit to the Isthmus which resulted in the discovery of the Pacific Ocean, is said to have been the first to prepare plans for a canal which it is claimed he did in 1529.

From that time on, through centuries, several nations—Spain, France, Britain, and America—made surveys and developed projects. Quoting history, "it made an inescapable appeal to popular, as well as to commercial and naval imagination." Nothing tangible or conclusive beyond surveys and discussions came out of it all, until the ill-fated de Lesseps' project was launched with loud acclamations in 1882. Fresh from the successful completion of the Suez Canal, and enjoying the prestige of its accomplishment, then of its nature the greatest in history, Count de Lesseps, operating under a concession of the then

proprietary Government of Colombia, undertook the digging of a sea-level canal through the Isthmus of Panama. He soon discovered that such an undertaking was vastly different from the one he had carried out at Suez.

The history of the failure of the French is of a time too recent and too fresh to need repetition here. As the inability to raise the funds needed for construction, owing to difficulties unforeseen and unprovided for, became more and more pronounced, this history, as I have studied it, was amply illustrated by the forced changes in plans in the famed Culebra Cut; first a change to a one-lift high-level canal, then to more locks and higher levels, all of which was plainly evident on the ground, and the contemplation of these changes was really pathetic. But with an ample supply of funds, their attempt to build any sort of a canal there, would have failed, as undoubtedly would the Americans have failed, until that then scourge of the white man in the tropics—yellow fever—had been mastered.

The de Lesseps Company was re-organized in October, 1894, and continued operations on a very limited scale, probably to enable it to hold its concession, until its rights and property were purchased by the United States under the authority of the Act of Congress of June 28, 1902. As soon thereafter as conditions and events permitted, active work of construction of a canal was begun by our Government, and pushed to a successful conclusion. Some of the interesting features of the initiating and executing of this gigantic enterprise have never been properly set forth, and it is my purpose here, to make them known without prejudice, conceiving that, as far as time will permit, a plain narration made by one who was intimately connected with the work during its earlier and formative stages, can only redound to the credit of the United States and to the vision of its engineers.

Under the authority of Congress, the President, in 1899, appointed a body of men known as the Isthmian Canal Commission, with orders to "investigate any and all practicable routes for a canal across the Isthmus of Panama, and particularly to investigate the two routes known, respectively, as the Nicaraguan route and the Panama route, with a view of determining the most practicable and feasible place for such canal." This Commission pursued the work of investigation as instructed, until as stated, the matter was closed by the purchase by our Government of the de Lesseps' rights and property at Panama. Then began the efforts of the Commission to round affairs up in shape so that plans could be made and the actual work of construction be gotten under way.

But by 1904, the Commission had amply demonstrated its inability to outline and to carry on any intelligent or progressive program of advancing the work, or of offering any hope for the completion of any kind of a canal anywhere, in any reasonable length of time, so its members were relieved and a new commission was appointed by the President.

The late John F. Wallace, Past-President, Am. Soc. C. E., had been the Chief Engineer for a year or more under the old Commission, and remained as such under the new Commission, until the early months of 1905, when he resigned the office and left the Isthmus. Mr. Wallace was a man whom I had

known for years, and esteemed, not only as a warm personal friend, but also for his great ability as an engineer. The statement is made here, that his action in resigning, under the conditions and the handicaps of the old Commission, was perfectly justifiable. His zeal had been dulled, and as he expressed the situation to me, he had no hopes that the new Commission would prove any more effective than the old one.

No one who knew him could justly impugn his high character or his ability as an engineer, and if any comment could fairly be made as to his conduct, or of his action in resigning, it would be that a more strenuous attitude toward his superior officers, would have been more efficacious and would have tended toward a more active state of affairs. Mr. Wallace was not an aggressive man, and there are times and conditions when fighting becomes a righteous duty. The situation which had developed at the date of his resignation, was such as demanded a one-man control, control by a man who was heedless of his technical reputation; and one only bent upon smashing a way through all obstacles, a kind of politic "roughneck", who did not possess too deep a veneration for the vagaries of constituted authority.

Whether I filled the above specifications or not, against my own inclination, I was appointed to the office of Chief Engineer, succeeding Mr. Wallace. My acceptance of the position was brought about by the persuasive tongue of William Nelson Cromwell, whose reputed good work secured the possession, by the United States, of the French rights and property on the Isthmus. It was plainly intimated to me that the Administration was gravely concerned as to what the reaction of the American public would be, if another failure to build the canal took place, and what the effect would be upon its own fortunes; so a final refusal to accept the position became impossible, and accordingly such appointment was made.

Previous to my departure for the scene of my future labors as the new Chief Engineer, I had a conference with President Roosevelt at Oyster Bay, on which occasion I laid down certain conditions, as nearly as conditions can be made to the President of the United States. One of these was that, while technically I would be under the orders of the Commission, I was to have a free hand in all matters, and would be hampered by no one in authority, high or low. Another one was, that I would not agree to remain until the completion of the canal, but would remain until the failure or success of the undertaking was assured according to my own judgment, and I fulfilled my promise in regard to the latter result, to the very letter.

The President made no objections to these statements, simply saying that the Canal must be built, and that he depended upon the new Chief Engineer to make it a success. In no way whatever at any time, did he give orders as to how matters should be handled. He made some suggestions, some of which were good, and some which were not adopted. He gave one intimation at the conference mentioned, and gave it in his characteristic manner, by an anecdote. A certain man suddenly became wealthy and set up a large establishment as a home. When his butler arrived he said to him, "I don't know in the least what you are to do—but one thing I *do* know, you get busy and buttle

like Hell!" He made some comments upon the old Commission which it is not necessary to quote here. He said furthermore that he did not know how affairs were, down on the Isthmus, but he thought they were in a "devil of a mess". When I reached the Isthmus, I found that the President was correct to a superlative degree in his opinion.

The President advised that I should address him directly in regard to matters connected with the work that I was about to undertake, and when the point was made, that such action might raise conflicts with the Secretary of War, or with the Commission, he brushed the objection aside, saying that all parties involved understood his wishes in the matter. As a consequence these wishes were carried out to a remarkable extent as far as information was concerned.

All reports that were current during the early period when the staff on the Isthmus was being organized, as to political influence in the selection of its personnel were false, as far as I was concerned. No such attempts were made, no officer of the Government and no member of Congress sought such favors for relatives or friends, either directly or indirectly. I was entirely free to choose my subordinates according to my judgment of their fitness.

An attempt was made by the head of the American Federation of Labor to intimidate me into granting special conditions for a certain kind of Union labor, in fact, to make a closed shop; and when such demand was rather forcibly refused, he threatened to take the matter to the President, and was told that he could take it to the Lord, if he chose, but that it would make no difference, and to close the door when he went out, which he did at once, and nothing further was heard on that subject.

The marine equipment of the Panama Railroad which had been taken over by the Government, included five small steamships in service between New York and Colon. These ships had just been fitted with refrigerator space and wireless outfits, the latter then only just coming into use. As practically all food supplies needed for the white employees on the Isthmus must come from the United States, a contract was made with a Chicago packing house for a regular supply of fresh meats, the prices for the same to be on a sliding scale, to be regulated by the wholesale prices as posted daily in Chicago. It may be said here, that such an arrangement proved very satisfactory in providing a regular supply of the best quality of everything, and at prices to the Isthmian employees at about 60% of the retail prices for the same meats right in Chicago.

After seeing that such necessary matters were properly attended to, I sailed for the Isthmus in company with the Chairman of the Commission and others, the Chairman remaining there only a few days, as health and other conditions did not seem to appeal strongly to him.

The first few days after arrival were devoted to getting as close a view of the situation as possible. It did not take long to realize that the discouraging reports current in the United States had not been overdrawn, but rather understated. There was no real organization, no competent directing head of affairs, few laborers, and what small amount of work was in progress, was in

most cases merely colorable and of little permanent value; but one outstanding fact was plainly evident and that was, that the problem of technical engineering must be relegated to the background until the fundamentals had been established; until something like a working organization had been created; until intelligent and comprehensive plans had been made and put into execution to collect a substantial working force; and, above all, until there had been established a proper morale and a belief that the Canal could and would be built, something that was lacking. The contrary belief was not wholly confined to the Isthmus, as before leaving the United States, otherwise intelligent and well-meaning friends encouraged the new Chief Engineer, by telling him how sorry they were that he had been picked as the "goat," as no one could build a canal at Panama. An optimist at that time was a rarity.

The great overshadowing cloud which hung over the enterprise, was the need of sanitation, and the crux of its needs was the all-compelling, absolutely essential requirement of clearing the Isthmus of yellow fever and of keeping it clear. While the situation was in a measure psychologic, there was a real grave danger which could not be ignored, and which would have imperiled the very life of the undertaking, had it not been removed. Whatever could be done to change the situation and restore confidence, must be done at once, done effectually regardless of method, cost, or personalities.

One of the first men that I had the good fortune to meet there, was Colonel (afterward General) Gorgas, the Chief Sanitary Officer, who at that time was reporting to the Governor of the Canal Zone. Literally, he was being supported in his tremendously important and vital work by no one. Colonel Gorgas was one of the finest characters that I ever knew, and it is a source of gratification to remember the friendship which resulted from close association with him, something more than official.

Like probably many others I had gained some little idea of the mosquito theory of the dissemination of yellow fever, by reading of the experiments of Italian scientists on the Roman Campagna, and had known something of the work of Walter Reed, Col. Gorgas, and others in Cuba, but, like most laymen, I had little faith in its effectiveness, nor even dreamed of its tremendous importance. But as should always be the case in such matters, an open mind was held on the unfamiliar subject, and it was well that the matter was so approached, for it seemed simply an act of Providence that just at that time such men as Gorgas and his very able assistants were on the job.

Several conferences were at once held with Col. Gorgas, who patiently and fully explained conditions, and what could be done to mitigate them, if only he could be given the means and support to do so. It was plainly evident from observation, that he had an efficient, loyal, and enthusiastic staff. Also, that he had very scant hospital space, practically no field working force commensurate with his needs, and that a greater part of his forces, skilled and unskilled, were quartered in makeshift houses mostly those of the old French régime. That, worst of all, a policy, not only of neglect but of absolute indifference and derision was openly expressed as to the value of the mosquito

theory, and it was being carried on by his superior officers, on whom he had relied for support.

I myself heard expressions from these officers, such as how "that crank" was wasting money in trying to drain water pools, in cutting grass and weeds, and insisting that every house and quarter should be screened against mosquitoes. Finally, the Chairman of the Commission advised me, that he had decided to eliminate Col. Gorgas—"to fire him", as he elegantly expressed it—and that he proposed to put a young osteopathist whom he had brought with him from the States, in Gorgas' place. My reply to this plan was more forcible than elegant, and perhaps would not bear repeating here and if paraphrased would lose most of its picturesque punch, but it had the merit of being effective, and Gorgas remained to carry on his wonderful work until the completion of the Canal. He and his staff were the men for the place, but if he and they had not been given the support when and how it was, as I shall detail later, it is likely that his work would have gone for naught, with more than a probability that another dismal failure would have been recorded and we would have had no Panama Canal to-day.

As related by Col. Gorgas, he had received little or no support from either Commission, the Chairman of the former and some of its members openly deriding his efforts, but his courage was an inspiration, and he thoroughly converted me to what was the truth—that he was on the right track to make a success.

So casting aside all question of existing authority or matters of official etiquette, cutting red tape entirely, I told him that the whole weight and resources of the Engineering Department, men, material, and money, were at his command. Laborers at this time were pitifully few in number, but they were turned over to the Sanitary Department as fast as called for, and other work, however important it seemed to be, was made to wait. The work of building houses and other living quarters, additional hospital accommodations, and of providing all classes of structures occupied by human beings with modern sanitary fixtures and screens, was pushed as fast as material and men to handle it could be obtained.

In a comparatively short time the good results of the work of the Sanitary Department became so apparent that most of its critics were stilled, even if some of them were not converted and failed to share the enthusiasm of the Chief Engineer. But it should never be forgotten, that the Engineering Department played a great and very vital part in the sanitation of the Isthmus, which made the success of the Canal possible. It was shortly after this time when Gorgas, at the suggestion of the Chief Engineer, was made a member of the Isthmian Canal Commission.

An interesting incident occurred some years after my connection with the work had ceased. President Roosevelt had been succeeded by President Taft, and the former had his office with the Outlook Company in New York. One day word was brought to me, that Col. Roosevelt wished to see me, consequently a call was made upon him.

After some pleasant remarks, Col. Roosevelt said that the trouble between Col. Gorgas and Gen. Goethals had evidently reached a climax, and that one or the other would have to go, and likely that one would be Gorgas. He said that he (Roosevelt) was well aware that I knew as much or more of Gorgas and his work than any man, and asked if he was the man for the place, and if he should be kept. I knew little of the merits of the controversy, but I did know Gorgas. I could barely find words to express my appreciation of Gorgas and his efficient work, and what the Sanitary Department under Gorgas had done, to enable me to win out in the face of what had appeared to be insurmountable difficulties, and in closing, I said that "if every other official on the Isthmus were called off, Gorgas should remain, and that his removal would be a stupid blunder."

Col. Roosevelt, pounding on the desk in his usual vehement manner, exclaimed, "By George, that settles it, Gorgas stays." And stay he did, until the completion of the Canal.

It has been a source of wonderment since that occasion, bearing in mind the not altogether harmonious relations which were reported to exist at that period between President Taft and Col. Roosevelt, how the latter had the influence, or power it may be called, to keep Gorgas where he belonged, at the head of the Sanitary Department of the Canal Zone. It was fortunate that he had such power, if he used it.

Below is quoted verbatim, a letter received from Gen. Gorgas, shortly after his appointment as Surgeon General of the Army:

"WAR DEPARTMENT
OFFICE OF THE SURGEON GENERAL
WASHINGTON

"APRIL 17, 1914.

"MR. JOHN F. STEVENS
875 Park Avenue
New York City

"DEAR MR. STEVENS:

"Yours of March 8th is acknowledged. I have a very clear and grateful recollection of the support and friendship which you always gave me on the Isthmus. I know very well at the time you refer to that you were the only one of the chief officials who believed in the sanitary work we were doing, and who was not taking active measures to oppose us. The fact is that you are the only one of the higher officials on the Isthmus who always supported the Sanitary Department, and I mean this to apply to the whole ten years, before and after your time. So you can understand that our relations, yours and mine, stand out in my memory of the trying ten years I spent on the Isthmus, as a green and pleasant oasis.

"I am glad your health is good and you are all right. I have just gotten settled in my new work here in Washington and look forward to the coming years with pleasure.

"Mrs. Gorgas joins me in kindest regards to yourself and Mrs. Stevens.

"Sincerely yours,
"W. C. GORGAS."

Needless to say that I treasure this letter as one of my most cherished papers. Considerable space has been given here to the sanitation of the Isthmus, without which the Canal could not have been built, so that the members of the American Society of Civil Engineers may realize that something other than technical engineering was done there during 1905 and 1906, by members of this great Society.

With sanitation assured and a well grounded faith that yellow fever would soon become a thing of the past, the way was clear to form an organization commensurate with the importance of the project. Enough details of such an organization had to be provided to care for the conduct of the work for a period of years, no matter what type for the proposed Canal was chosen. No time was lost in waiting for the results of sanitation to become apparent, for within a short time after reaching the Isthmus the Chief Engineer felt well assured that success was fully made certain. There was already in service first-class material sufficient to form the nucleus of an organization. All it needed was faith in a leader, whom it could trust and follow with confidence.

The various classes of the work, as far as then could be seen, were segregated, and selection was made of what appeared to be the best men to head each department, and almost without exception the selections proved to have been wisely made. One general policy was adopted and strictly followed, and that policy faithfully pursued is the only one that leads to efficient administration, it is to give an official ample authority, and then to hold him responsible for results. A maxim laid down, and which soon was thoroughly understood, was that if a man was right three times out of four, he was a success, but that if he wanted to make his failure certain with the Chief Engineer, he should do nothing of his own initiative, but "pass the buck".

The personnel was strengthened from time to time by additions to its members, and it was not long before an organization was built up, sufficiently strong and yet flexible enough to meet all demands. Considering the magnitude of the undertaking, its uncertainty at first, the condition into which it had been allowed to drift, and its location in a tropical country, the statement is justifiable that never in the history of construction projects was any organization better fitted for its purpose.

The lack of a decision as to what type of a canal should be built, was not so much of a handicap as might be supposed. Specifications were made, and plant material and supplies were ordered in vast amounts and of endless variety. And it was very satisfactory later to know that not a thing was gotten but what was well adapted for, and was used in the progress of, the work. And much of such work was in progress and the greater part of the plant provided, before the type of the canal was finally decided upon.

The discussions began as to whether a so-called sea-level (a misnomer by the way) or a lock type of canal should be adopted. The alluring phrase "sea level" naturally caught the fancy of the multitude, and after a majority of the International Board of Engineers, which President Roosevelt had convoked, had reported in favor of such a type, the matter seemed to be settled. But some of our most competent engineers who had given the matter close

study, were of the opinion that the lock type, considering all the factors which entered into the problem, was eminently the proper solution, and the key to the proposition was the Gatun Dam.

This dam had been suggested some years previously and it appealed to the Chief Engineer as a certain means of curbing the unruly Chagres River and making it the servant, instead of the master, of the situation, and he became an ardent advocate of the lock type, such as was ultimately selected, and which is proving so successful in operation.

The President had an open mind as to the choice of type, probably he rather inclined to the sea level. The Chief Engineer had several conferences with him and with the Secretary of War, and argued to the limit of his ability in favor of the lock type, which the President recommended to Congress as the proper type to be selected.

In a report as of date January 26, 1906, which the Chief Engineer made to the Isthmian Canal Commission, the following language was used:

"The sum of my conclusions is, therefore, that, all things considered, the lock or high level canal is preferable to the sea level type, so called, for the following reasons:

"It will provide as safe and a quicker passage for ships, and therefore will be of greater capacity. It will provide beyond question, the best solution of the vital problem of how safely to care for the flood waters of the Chagres and other streams. Provision is made for enlarging its capacity to almost any extent, at very much less expense of time and money, than can be provided by any sea level plan. Its cost of maintenance and fixed charges will be very much less than any sea-level canal. The time and cost of its construction will not be more than one-half that of a canal of the sea level type. The element of time might become, in case of war, actual or threatened, one of such importance that measured not by years but by months and even days, the entire cost of the canal would seem trivial in comparison. Finally, even at the same cost in time and money for each type, I would favor the adoption of the high level lock canal plan, in preference to that of the proposed sea level canal. I therefore recommend the adoption of the plan for an 85-foot summit level lock canal, as set forth in the minority report of the Consulting Board of Engineers.

"Very respectfully,

"JOHN F. STEVENS,
Chief Engineer."

The report of the Commission to the Secretary of War, and the latter's report to the President, embodied these points, and the President in his message to Congress recommended the adoption of the lock type. His recommendation was adopted by Congress, quickly by the House, but only after some delay and discussion in the Senate, by the small majority of four votes, small, but enough. The able speech of Senator Knox on the floor of the Senate, in favor of the lock type, was probably the deciding factor, but without violating any confidence it may be said here, that much of the speech was read directly from type-written sheets dictated by the Chief Engineer.

Soon after the type of the proposed canal had been decided, the question arose as to how, that is, by whom should the work be carried out, directly by Commission forces or by contractors. It was rather felt that strong contractors who had trained organizations and large personal followings, could better

handle such enormous work than could a new and untried organization, and that it would be better not to have the Government go into such business directly, and the Chief Engineer was at first rather inclined to sympathize with that belief. Accordingly, he was asked by the Commission to submit a form of contract for its consideration, which he did.

Owing to the great multiplicity of details involved, it was manifestly impossible to ask for bids upon unit quantities, so, using as a basis a modified form of percentage contract which he had successfully employed in heavy railroad construction work, and which had been passed upon by competent legal authority as legally air-tight, he prepared a form, modified and hedged about in such a manner as to make the Chief Engineer supreme in all matters and questions that might arise, for the chances of such were numberless.

The discussion of this proposed form—of asking for tenders—was taken up directly by the President at a Cabinet meeting at which the Chief Engineer was present. It met the tentative approval of the President and members of the Cabinet excepting that of Secretary Root, who insisted upon injecting some conditions which practically destroyed its value, but of course his views prevailed, and the Chief Engineer left for the Isthmus knowing full well what the result would be.

Tenders were invited and received from several syndicates, and a summary of the bid which was considered most favorable, was cabled to the Chief Engineer, who, after due deliberation, cabled his disapproval, with a recommendation that the Commission carry out the work directly itself through its Engineering Department. This report was approved by the President, and all idea of letting a contract was dismissed.

The reasons which caused the disapproval of the bid by the Chief Engineer were several in number. One was that, in view of the magnitude of the enterprise, the percentages asked were too large. Another reason was that the contract form had been so emasculated by the introduction of Secretary Root's changes, as to make it practically impossible to carry out the work without endless friction and conflicts of authority. Another reason, and not the least, was that the Chief Engineer had no confidence in the administrative ability of the parties with whom he would have to deal if the contract was awarded to them. And so the work was carried through by the engineers, and its execution, both from a technical and business standpoint, forms one of the brightest pages in the history of construction projects.

It will be noted that the sea-level type has been designated here as, "so-called" or "proposed". In explanation, please note that no operating ship canal at the level of the oceans could function, without the introduction somewhere in its length, supposedly near the Pacific entrance, of a tidal lock, unless the canal was of a much wider section than was recommended by the majority of the Consulting Board of Engineers. The difference of the tides at the Atlantic and Pacific ends is generally 20 ft. and often more than that.

As recommended by the majority of the Consulting Board, the sea-level canal for 21 miles would have had a bottom width of but 150 ft., without turnouts, and at the depth to which it would have been dredged, with rough,

rocky sides—a narrow, tortuous, crooked waterway, with thirty torrential side streams pouring down precipitous slopes into its section, in a country where 1½ in. of rain per hour has been recorded for 10 consecutive hours. No speed that would permit of steerage way could have been maintained under such circumstances in such a ditch, and the slightest impact of a large ship against the rocky sides, would have crushed her plates like eggshells. Competent, experienced shipmasters have assured the speaker that no sane navigator would have risked his ship in such a channel.

It was proposed to hold back the Chagres River by a dam 180 ft. high to be built at Gamboa, directly alongside the Canal, and the overflow from the river was to be taken care of by regulating works at the dam. With the storage basin completely filled, as it would have been in the rainy season, imagination can picture what the effect of 150 000 cu. ft. per sec. of flood waters (which has been recorded) would be when poured directly into the prism of the Canal through those regulating works.

The plans proposed for taking care of the flood waters of the smaller streams, were equally futile and impracticable. At no place was the sea-level canal proposed to be more than 200 ft. wide, and taking it as a whole, even if it could have been navigated at all, by ships of any considerable size, which is doubtful, it would have been at such slow speeds as to make it practically a makeshift. As contrasted with the present lock type canal, with the latter's 1 000-ft. approaches, its 30 miles of Gatun Lake where any speed can be maintained, and with only 8 miles of a minimum width of 300 ft., there is no room whatever for argument. The high-level lock canal can and does pass ships of any size thus far afloat, much quicker than could such a sea-level canal as that proposed, even if the latter could have passed them at all, which is extremely doubtful.

It is not argued that granting a *true* sea-level canal were built, even with a tidal lock, of a width sufficient to allow a fair rate of speed for a ship moving through it, and always providing that the flood waters of the Chagres and other streams could be safely taken care of, that such a canal would not be superior as a method of transit between the two oceans. But the time required to build such a canal, and the enormous cost of it, as truly said by President Roosevelt, rendered it prohibitive. No mistake was made in the adoption of the type of canal that we have to-day.

The vulnerability of the lock type in case of an attack by an enemy has been urged against it. This criticism is quite true, and the same is true of such a sea-level canal as was proposed. Its one tidal lock would have been equally open to damage as are the locks of the present canal, and the destruction by bombing of the 180-ft. high dam at Gamboa, letting some 200 000 000 to 300 000 000 cu. ft. of water smash into the canal, would not be a pleasant thing to contemplate, and the sea-level canal as proposed, with its narrow, crooked channel, could have been easily blocked by the sinking of a ship. No canal can be made safe from damage by an enemy in war time, excepting by keeping him away from it. It must be remembered that twenty-two years ago,

bombing planes were in the future, and even so, the lock type is no more vulnerable than a narrow sea-level canal would have been.

Once the lock type of canal had been decided upon, the way was cleared so that the plans incident to its construction could be made and put into execution. The excavation of the immense amount of material in Culebra Cut was already under way and going rapidly, for, of course, it was evident that millions of cubic yards could be removed, whatever type was to be chosen. Many wash-borings had been made in the valley of the Chagres River at Gatun, and these borings proved conclusively that a stable foundation for both dam and locks was there, contrary to the vehement assertions of the sea-level advocates. As before remarked, the Gatun Dam was the keynote, the very motif of the lock type, and its adoption and construction was the great engineering accomplishment of the enormous project of the Canal. It is, of course, true that hundreds of other important engineering problems remained to be solved, but they were more in the nature of routine, and precedents existed to guide in their solution. Gatun Dam and Locks constituted one controlling feature in the time and success of building the Canal; and the other one was the excavation of Culebra Cut, and this was primarily a transportation problem, and to a much less extent, one of engineering.

Time and space do not permit the detailing of the many—it would be complimentary to call them crack-brained—schemes, which were urged upon the Chief Engineer to move these millions of yards. All such schemes were waived aside, for he knew from experience that the work could only be done in a reasonable time, and at allowable cost, by steam shovels and work trains. Material, rock or earth, can be excavated quickly, but the problem of transportation is not so easily solved. The dipper must be kept moving every possible minute, if the steam shovel was to do the work. Accordingly, the Chief Engineer devised a system of trackage, complicated from a layman's point of view, but really extremely simple, which permitted the minimum loss of time in moving the empty cars to the shovels, and in taking away the loaded ones. That was all there was to the problem. Its solution was a success, so much so that the system was continued until the entire mass of material had been removed from the Cut, and the end of work came in the very year when the Chief Engineer had predicted that it would come, and he did so in the face of criticisms and even ridicule from doubting engineering and transportation experts.

Meantime, to secure and maintain a sufficient amount of labor, skilled and unskilled, employment agencies were opened in many of the large cities of the United States. Agencies for the recruiting of common laborers were established in the British and French Caribbean Islands, and the necessary shipping arrangements were perfected. There seemed to be an endless supply of blacks to fill the requirements for common labor, but very soon it became painfully apparent that these people had gotten the totally mistaken idea into their woolly heads, that no other than negro labor could be had, and their efficiency, if such a word could be applied to them, began to decrease to an alarming degree. It became apparent that they needed an object lesson and they got it.

At first, a proposal was made to bring Cantonese coolies over from China, but this the Chinese Government, in view of the de Lesseps' experiment, would not permit. Then an attempt was made, or suggested, to get laborers from Cuba, but this raised the protests of the sugar planters and had to be abandoned. Finally, the Chief Engineer sent an agent first to Spain, from which country he was promptly thrown out, and then to Paris. From his office there, he was able, by some *sub rosa* negotiations with the steamship lines, to send over about 7 000 peasants from the Biscayan Provinces of Spain.

These men were hired at the rate of $1.60 gold per day of 8 hours, just twice what the blacks were receiving. In a few days a number of the Spaniards struck for $2.50 per day and proceeded—more or less armed—to clean the blacks out of Culebra Cut. They were met quickly by a force of the Canal Police, and a conflict was staged, during which several of the Spaniards achieved immortality, and then the mass of them went back to work, and little or no trouble was had with them thereafter. A stern lesson was needed, for if they had succeeded in terrorizing the negroes, every one of the latter would have taken to the jungle, and the labor and organization of months would have been destroyed.

All this time houses and quarters, hotels, school-houses, churches, jails, of course, warehouses, and docks, and many other classes of structures were being built, about 75 000 000 ft. of timber, mostly brought from Puget Sound, being used in 18 months. The Commissary, operated under the Panama Railroad, was thoroughly re-organized, and constantly carried a stock of every conceivable article, of a current invoice value of nearly $1 000 000. A system of cash payments—through coupons—was put into effect. Delivery by railroad supply train to practically every family and eating place on the Zone was completed by 9:00 A. M. of each day. A refrigerating and ice plant, a laundry, and a bakery were installed in connection with the Commissary, and the employees could buy anything in the line of food from beef and pork to ice cream, and luxuries from evening clothes to face powder, and all at exact cost to the Commission, which generally meant at about 70% of the prices prevailing in the United States. Meals for the whites, such as did not keep house, cost thirty cents, and these meals equalled those furnished in this country at double the charge. It was about at this stage of proceedings, that the Chairman of the Commission sent to the Zone a party who had been operating boarding houses for laborers on some of our Western railroads. He brought a proposal approved by the Chairman, that he would take over the Commissary, and the feeding of all classes of employees, and as far as the price of meals was concerned, it was suggested that they be fixed at 33⅓% above those already in force, and, presumably, a similar approximate advance on other Commissary supplies. This brilliant business proposition was instantly vetoed and the Chief Engineer continued to administer the food supplies and the Commissary through the proper department.

Colon and Panama were provided with pure water, and with modern sewerage systems, and each city was paved to a large extent with vitrified brick brought from Illinois. Some twenty or thirty small towns were built along

the line of the Canal, and all were provided with modern sewerage and water systems, hard surfaced streets, sidewalks, etc. In the Canal prism, an 8-mile air pipe line was laid, supplied by two large compressor plants, to give air to the hundreds of rock drills used in Culebra Cut.

Much concrete was used, the aggregate for which was furnished by crusher plants located at convenient places. A location was secured from the Panamanian Government, about twenty miles along the coast from Colon, where an immense gyratory crushing plant was installed to furnish the crushed rock for the Gatun Locks and other construction at that end of the Canal. Engineers experienced in such work were placed in charge of the design of the locks and their many adjuncts, and this important feature of the project was well under way in the early months of 1907, these same engineers continuing their tasks throughout the entire time of construction of the Canal.

It would be futile to further attempt to name the many other necessary accessories which had to be designed and built to carry on the construction work of the Canal. The recital here of all these activities may seem tedious, but they are briefly touched upon because they were carried out mostly by members of the American Society of Civil Engineers. It will be recognized that engineering as carried on in the multiplicity of requirements on the Canal Zone, was a very comprehensive term. It was a truly paternal job for the Chief Engineer. One hour he would be studying and deciding on some important matter, and the next hour trying to soothe the grief of some matron who complained that so and so, the wife of another employee who received the same salary as her husband, had one more rocking chair than she had, all quarters being furnished.

The work of rehabilitating, double-tracking, and otherwise enlarging the facilities of the Panama Railroad, which, in addition to moving a large commercial business, was utilized to handle all work trains, including the hundreds in service at Culebra Cut, was in progress. The management and operation, on the Isthmus, of the Panama Railroad Steamship Company came under the authority of the Chief Engineer, as also did the handling and stevedoring of the ships of some dozen of foreign lines.

An erroneous idea long prevailed and likely does yet, with the general public and some engineers, that the work of these two years covered only that of preparation—far from such was the case. While the work of preparation was being pushed, so also was the actual construction of the canal proper. The dredging of the ocean approaches, the excavating for the locks at Gatun and Pedro Miguel, the construction of the dams at both places, was actively proceeding. Many steam shovels, drills, and work trains were successfully at work in Culebra Cut, so that during the early part of 1907, the material removed from the Cut averaged about 750 000 cu. yd. per month. The whole elaborate machine which had been built up so quickly and carefully, was working smoothly, and the output of all classes of construction was increasing daily at a rate that exceeded even the sanguine expectations of the Chief Engineer. The hardest problems were solved, the Rubicon was crossed, the Canal was being built, and everything was set for its completion in the time pre-

dicted by the Chief Engineer. Only gross mismanagement or a failure to supply the necessary funds, could militate against its triumphant accomplishment.

Much to his surprise the Chief Engineer was appointed Chairman of the Isthmian Canal Commission, taking up its duties in addition to those already in his charge. This appointment was justly regarded by him as an expression of approval, on the part of the President, of the Chief Engineer's work and a recognition that the successful construction of the Canal was assured.

Not long afterward, knowing full well that there had been carried out—even beyond the letter—the promise made to the President and knowing that the latter was alive to the fact, I resigned from the service of the Canal. Many reasons, mostly from irresponsible scribblers, were given as the cause of this resignation. In one particular the reasons alleged were alike—they were all absolutely false. Not the slightest friction ever developed between the Chief Engineer and the President, or, in fact, with any one, high or low, connected in any manner with the enterprise. The reasons for the resignation were purely personal, and involved nothing in regard to the work nor with any parties associated in it.

I have never declared these reasons, and probably never will, as they were private and of no particular interest to the public. I have a petition signed by thousands of the white employees then on the Canal Zone, asking me to withdraw my resignation, and promising to work even harder to push the construction to completion; but it could not be. The expressions of regret which greeted me from the immense number who gathered to bid me farewell, a gathering which in size probably exceeded any function ever held on the Isthmus, were sincere and fervent. I was pleased to receive them, and to know that personally as well as officially, my work had borne fruit; and I felt well assured that the work which had been so near my heart, had been given into competent hands, as the future proved in every way to be the case.

Last February, it was my privilege and pleasure to visit the Canal Zone, the first visit since my resignation in 1907. While there I was the guest of Governor M. L. Walker and his charming family, and the hospitality and reception which I met from them, and from the entire entourage on the Zone, was overpowering. Governor Walker and I were friends of many years' standing, and it was no surprise to note the perfect manner in which the operation of the Canal and of its many adjuncts was being carried on, due to his administrative ability and his able staff. Receptions, dinners, and social functions were showered on me, and best of all, I was made to realize by high and low—by all classes—civil and military—that the part I was called upon to take in the creation of the Canal was not forgotten. It was an experience the recollection of which will linger as long as life lasts.

The daylight hours of those ten days were spent in examining in detail practically every feature of the Canal. Ocean approaches, marine and railway terminals, dams, locks, Culebra Cut, nothing of consequence was overlooked; and the general result was that the whole machine was the realization of a

dream of twenty years before, only the realization was more perfect than I had dared to hope.

It would be trite and quite unnecessary here to describe in detail the operation of passing ships through the locks. It is not much less than a marvel to see what perfect control is had over every movement, and how quietly and quickly those movements are performed. The locks fill and empty—each movement—in 7½ min., and it fills one's mind with something like awe, when he sees that by the movement of a switch, no larger than one's finger, a 25 000-ton ship rises, or falls as the case may be, as easily as one would juggle a jack-straw. First-class machinery and perfect co-ordination of all parts, with intelligent trained operatives, does the work, and forcibly demonstrates how man's engineering talents can make the mysterious forces of Nature the servants of man's will.

Much time was given to examination in detail of the locks, their chambers, gates, motors, signals, control towers, emergency dams, in fact, of all their parts. The process of locking ships through was fascinating to watch, the ease with which it was done and the silence of it all was impressive. A trip on a ship from Buenos Aires en route to San Francisco carried me through the whole length of the Canal. Also in company with Governor Walker, some hours were spent in a launch examining Culebra Cut. An interesting four hours was taken at Gatun Dam and Locks. Most engineers, at least know the dimensions of that dam—8 400 ft. in length, ½ mile wide (thick) at its base, and sloping gently to a width of 100 ft. at its crest. The top of the dam is 105 ft. above sea level, or 20 ft. above the normal level of Lake Gatun, which it forms. It is an earth dam, planted with grass and shrubbery, and has the unique distinction of being the only dam in the world having a golf course on it, a standard length nine-hole course. Employees refer to the dam as "our ten million-dollar golf course".

At the spillway is located the plant which generates the power and light for the Canal and its signals; light for all shops, offices, residences, and quarters; power and light for fortifications; in fact, for each and every electrical need in the Canal Zone. In addition, there is at Miraflores a very large installation of steam and Diesel electric generating stations, so that if trouble occurs at the hydro-electric plant at Gatun, a switch throws the entire load on the emergency plant at Miraflores with no delay whatever.

But all such descriptions are merely threshing over old straw, and it is needless to devote time and space to even a recapitulation of the most important features of the Canal, they must be seen and studied with an experienced eye to be appreciated. Suffice it to say, that in one man's mind at least, and his opinion is shared by thousands who are qualified to speak, it is as well nigh as perfect a machine, in design, in construction, and in operation, as the genius of man can plan and execute.

The work of Maj.-Gen. George W. Goethals, U. S. A. (*Retired*), M. Am. Soc. C. E., former Chief Engineer, and of his associates on the Zone, cannot be given too high credit. The details of the Canal and of its accessories have been wrought out and completed marvelously well. Nothing necessary to make a complete and perfect whole has been forgotten, and the character of the

work, as regards material and execution, is superb. No mistake was made in entrusting the work to their hands, and while others might have done it as well, I feel that they could not have done it better, and I take this occasion to pay a high and well deserved compliment to Gen. Goethals and his associates.

The average time of passage of ships through the Canal is 8 hours. Its generous widths would permit of shortening this time if occasion required. It can handle easily 48 ships per day, or more than 17 000 per year. It is 42 ft. deep in the Atlantic and Pacific sea-level section, from 45 to 85 ft. deep in Gatun Lake where it is 500 to 1 000 ft. wide, and 40 ft. deep and 300 ft. wide in Culebra Cut. It has large coal and oil storage plants, extensive marine shops, and one of the largest dry docks in the world. There are modern hotels, restaurants, laundries, ice and refrigerating plants, churches, schools, clubs, places of amusement, and Balboa, where the Administration Headquarters are located, is one of the loveliest places for residence that I have ever seen.

The Canal tolls are levied on the net tonnage of the ships and range from 25.7 cents upward, according to capacity and not the nature of the cargo, but of the class of ship. The average toll for bulk cargoes is about 65 cents per long ton. The cost to ships using the Canal is approximately equal to six days at sea; of course, this varies with the class of ship and her speed.

As a concrete example: A steamship passing through at the time of my visit, carrying passengers and a limited amount of freight, New York to San Francisco, tonnage, 22 500, speed about 14 knots, costing for overhead, depreciation, maintenance, and operation not far from $2 000 per day, saved about 8 000 miles, or some 23 days, by using the Canal instead of the Straits of Magellan. The tolls were $11 000, saving $35 000, besides which, that line would not be sending ships around the world carrying passengers if the Magellan route was obligatory.

Periodically, the capacity and the water supply for lockages of the Canal have been questioned, recently with an insistence and a demand which has already reached the President, and probably will reach Congress, that a canal at Nicaragua be built without delay to relieve the situation. Let us look into what, taking these demands at their face value, would indeed be a very vital matter. Thus far, the water supply has proven ample. Gatun Lake covers an area of 163 sq. miles and it is entirely practicable to raise its normal level 6 in. and possibly 1 ft., to elevations of 85.5 or 86 ft., which would add 52 160, or 104 320 acre-ft., respectively, to the present supply of water. The water-shed of this lake covers an area of 1 350 sq. miles, and from this immense water-shed there flows into the lake a number of large streams, on many of which storage reservoirs to conserve the flood waters which descend frequently, can be built.

But plans for a very large increase in the present supply were made long since, in fact, some years before the type of canal was selected. About 6 miles up the Chagres River, at Alhajuela, there is a natural site for a storage dam which would impound flood waters to an amount nearly equal to the present supply. This dam can be built in 3 years, or less, and would cost approxi-

mately $8 000 000 at present prices of labor and material. With the additional supply thus afforded, all fears of a shortage of water could be dismissed for many years to come. It is true that given a season when the rainfall is abnormally light, the necessity for conserving the supply is apparent. A large quantity of water is now used to create power to generate electricity, but in case of necessity this water could be saved by running the steam-generating plants previously described. Twice I visited the site of the proposed Alhajuela Dam in 1905, and made a thorough examination of its availability, and I know that it is wonderfully well adapted for the purpose planned.

As a measure of insurance, the construction of this storage dam should be begun as soon as it can be authorized, although the increase in traffic must be greater than it has been for some years, before any serious apprehensions need be felt on account of a shortage of water.

The Canal is operated only during daylight hours, but it can be readily operated during the night, as it is perfectly lighted throughout its entire length. It has been noted that the nominal daily capacity of the Canal is 48 passages, but on January 17, 1924, a total of 57 vessels made the transit. In 1923, a total of 5 466 ships passed through, and, in 1926, a total of 5 964, an increase in 3 years of 498, or an increase of 9% in 3 years, or an average increase each year of 3%, which does not seem to bear out the claims made by irresponsible parties as to tremendous increases in traffic.

The daily average number of ships passing through the Canal during May, 1926, was 15.9 and for 11 months, ending with April, the highest number passing in any one day was 16.3, the average number for that period being 14.97. Therefore the statement that the Canal is not and has not been worked to more than one-third of its capacity, is ultra-conservative, especially so when it is considered that 57 ships have passed through in 1 day without difficulty.

The capacity of this Canal should not be measured by dead tonnage, the number of ships passing through is the real yardstick. Copper-laden ships from Chile, lumber-laden ships from Puget Sound, and oil tankers from the Pacific Coast, all load heavily and, in locking, a ship is a ship in point of time, regardless of its tonnage.

The Sault Ste. Marie ("Soo") Canal at the outlet of Lake Superior, during the 234 days when it was open in 1926, passed 21 000 vessels of all sizes, or an average of 90 per day, and its tonnage for 1926 was about 86 000 000. About 92% of this large traffic is handled by the American locks, and the greater part of this through the third and fourth (two in number) locks. If the "Soo" Locks could work the entire year, as can those at Panama, they would, on the basis of 1926, pass 32 850 vessels, or about 134 000 000 tons. The Panama Canal passed 26 836 241 tons in 1926, exceeding that of the sea-level Suez Canal. In view of the ease with which the "Soo" Canal could pass more than 100 000 000 tons yearly, if not icebound for four and one-half months, what could the Panama Canal pass in twelve uninterrupted months, with practically the same facilities as the "Soo". And yet, reliable authority declares that the "Soo" Locks are not worked to more than 50% of their ultimate capacity.

The four-year period from 1923 to 1926 shows over the preceding period an increase of 87 per cent. The maximum number of ships passing through the Canal in any one month was 611, in March, 1927, or an average of 19.7 per day, or an average of not over 40% of its capacity.

Liberty is here taken of quoting Governor Walker, who is probably the best informed man in the world as to the actual situation:

"I do not think that we should take the increases shown above too seriously as a basis for predicting future traffic through the Canal. It must be kept in mind that, due to the World War, routing of all shipping was disturbed, and it was a matter of years before normal trade routes could be re-established. The phenomenal growth of traffic through the Canal between 1919 and 1923 can be largely attributed to this re-adjustment. It should be noted that the increase between 1923 and 1926, four years, is only 7.6% for commercial carriers, and it should further be noted that 25% of our business now is incident to the transfer of oil from the Pacific Coast of the United States to the Atlantic Coast, and that this may cease more or less entirely due to the production of oil on the north coast of South America, and a revival of activity in the Mexican fields."

But thus far, the whole story of capacity has not been told. The present double, or twin, locks of the Panama Canal were located and built so that when the need arose for greater capacity than they afforded, a third set of locks could readily be added alongside the present ones. Probably in some years to come it may be advisable to add these locks, which can be done without disturbing those now in service. The present locks are 1 100 ft. long, 110 ft. wide, with a draft of 41 ft. over the miter-sills. These locks safely and quickly have transited the largest, that is, the widest ship afloat, and can do the same with even wider ones. But probably it would be wise, as a measure of prudence, to design the new locks of larger capacity, and of deeper draft, although it must be remembered that when ships are built requiring more than 40 ft. of water, many of the most important harbors of the world must be deepened accordingly.

Again quoting Governor Walker:

"Since we are running now, excepting when we put the United States fleet through, at about 40% of our capacity, the third lock with additional water storage will very nearly double this capacity. I cannot see any future for the Nicaraguan Canal."

Again:

"In my opinion the correct policy now is to proceed at once with the construction of the Alhajuela storage project, to watch the situation and commence the installation of the third lock when indications warrant and to entirely dismiss the Nicaraguan Canal from consideration for a long time to come."

In which opinion and conclusions I, who have watched and studied the situation closely for the past four years, most emphatically agree. The more than $500 000 000 which a canal at Nicaragua would cost (and, by the way, it would be a lock canal requiring a lift 30 ft. higher than the present Panama Canal because Lake Nicaragua, the key to the situation, is 115 ft. above sea level), would practically be wasted, and if spent at all, had better be spent

in our country where vast problems confront us, and with which only National resources can cope. The addition of the third lock and the increased water supply at Panama which the storage dam at Alhajuela would afford, as far as human eye can look into the future, would give the Canal ample capacity for all transits for the next fifty years, and perhaps longer.

To any one who thinks that the capacity of a lock canal is very limited, a study of the "Soo" Canal would be profitable. This canal carries the great bulk of the tonnage that passes Detroit, and yet three years ago (later data are not available) more tonnage passed Detroit in eight months of open navigation than the total of all tonnage that passed through both the Suez and the Panama Canals in two years.

As previously remarked in discussing the relative merits of a sea level *versus* a lock canal, I am in the least arguing that a lock canal is superior to a real, *bona fide* sea-level canal, such a one as our erratic engineering friend, Bunau-Varilla—one of de Lesseps' Chief Engineers—termed the "Straits of Panama". And speaking of Bunau-Varilla, one is reminded of the definition of a philanthropist, one "who is always ready to spend the other fellow's money."

There has always been a mistaken idea with the general public, as well as with some engineers, about the so-called sea-level canal as proposed by the majority of the Consulting Board. If built, it would have been a useless, expensive toy to be flung sooner or later on to the National scrap pile. I have been privileged to be of some little service to my country, and the greatest service I ever gave to it, was the part I took in preventing foreign votes from foisting a nameless, useless thing upon a too credulous American people.

Probably the most of the critics of the present Canal are sincere in their criticisms, but many of their criticisms show a decided lack of knowledge of conditions. As far as the danger to it from earthquakes is concerned, Mother Earth is a nervous old lady, and when and where and how hard she will wiggle the next time is beyond the ken of the wisest, as she is probably wiggling somewhere all the time. The United States is not free from her vagaries. The Pacific Coast at San Francisco, the Atlantic Coast at Charleston and in New Jersey, the mountain ranges of Montana, and even staid old New England, have felt the rattling of her old bones. Panama may have quakes, and like most parts of the earth she probably will have, but it is reasonably certain that no severe ones have occurred there during the past 200 years, or some fragile masonry structures known to have been built even longer than that, would not now be standing practically as they did when their long-forgotten builders completed them.

Ninety-nine times out of one hundred when the words, "Panama Canal", are spoken, the question is asked, "How about the slides?" This same subject has been the windmill against which zealous Don Quixotes have been tilting for the past fourteen years. That there have been serious slides is well known, and it is within the realms of possibility that there may be others. As noted previously, I took occasion during my recent visit to the Canal, to examine Culebra Cut, and my opinion is, that while slides may occur, there was not to my eye any reason to believe that they would be serious enough to block the

passage of ships, for it would take a very large slide to close the 300-ft. wide channel which exists throughout the whole length of the Cut. Enormous dredges with ample other floating equipment are at hand, and the channel is kept clear. Criticism is heard that the slopes of the Cut should have been made flatter—this is a matter of opinion. It is a well-known axiom among railroad construction engineers that, in a cutting where the material is earth, or earth and rock, especially where the rainfall is heavy, the steeper the slope can be safely made, the less cost of maintenance.

The flatter the slopes in such cases, the more area is exposed to rainfall, and the more loose material is washed into the cutting. And at Panama, where the rainfall is always heavy, where torrential rain has been known to fall at the rate of 1½ in. per hour for 10 consecutive hours, this was a factor to be considered. I do not know what determined Gen. Goethals to use the slopes that he did, but if he decided upon them as the result of his best judgment, he did what was exactly right and just what I would have done if I had been responsible for results.

There is another angle to the situation. Granting that approximately as much material has been taken out of the Cut since the opening of the Canal, as was taken out of the original prism during construction, it is probable that the removal of this additional yardage has cost much less, possibly only one-half as much as it would have cost if taken out before the opening of the Canal. Dredging is one of the very cheapest methods of handling such material.

There are some who criticize the sun, which is now almost at the peak of one of the periodic phases which come upon it every eleven years. One of the symptoms is an increase in the number of spots; another is a slight increase in the average heat of its rays; another important symptom is that the percentage of the ultra-violet rays is far above normal. But the sun keeps right on shining and fructifying the earth, giving us everything that we have, even life itself, and will probably continue to do so for some time to come.

The three Chief Engineers who were responsible for the planning and construction of the Panama Canal, were Members of the American Society of Civil Engineers, as were also many of their subordinates. The Canal is very largely the creation of this Society, and as such its members should take a just pride in it, as being a marvelous triumph of man over the hostile forces of Nature, and a work which, from an engineering and business point of view, can truly be classed in a rank by itself, a rank which has never been reached before since the dawn of civilization.

And in closing this sketchy review of a few of the activities in which I had a part, and particularly in consequence of the demands which are being made for something different, unnecessary, and something which the most of its proponents do not themselves understand, we may call to mind and apply the words of wise old Omar, the Persian poet philosopher:

> "Some for the Glories of this World; and some
> Sigh for the Prophet's Paradise to come;
> Ah, take the Cash and let the Credit go,
> Nor heed the rolling of a distant Drum."

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

TIDES AND THEIR ENGINEERING ASPECTS

By G. T. Rude,* M. Am. Soc. C. E.

To Be Presented September 7, 1927

Synopsis

In the formulation and support of many of his theories and conclusions the scientist generally is compelled to use evidence of a qualitative nature, which later may be open to dispute when further qualitative evidence of contrary indications becomes available. The engineer, however, requires quantitative data of indisputable character and seeks continually for the accumulation of data and material of a quantitative nature, on which to base his conclusions and specifications for future operations. These data may be the result of innumerable tests of material, of structural weaknesses, etc., from which finally mean values of almost unerring certainty are obtained; or they may result from the accumulation of records of natural phenomena of periodic or seasonal character. In consequence of this continual quest, for example, long series of observations of tides and currents have been made throughout the world.

While the primary object of these observations has been to furnish data for advance predictions of the tides and currents at any future date for the use of the mariner, within recent years the needs of the engineer have been given consideration. As a result comprehensive current and tide surveys are now being made by the United States Government of all its important harbors. The data from these surveys should prove of considerable value to the engineer engaged on harbor and river development and in marine construction of all kinds. These systematic surveys were begun in New York Harbor in 1922, followed by others in San Francisco in 1923, in Delaware Bay in 1924, and in Boston and Portsmouth in 1926.

To the engineer an accurate record of the rise and fall of the tide over a considerable period is not only of advantage but generally necessary to a

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in **January, 1928,** and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

* Commander, U. S. Coast and Geodetic Survey, Washington, D. C.

proper understanding of the conditions under which operations will be carried on, or of the physical conditions which will result from an engineering project. As an example of the latter the reconstruction of the Chesapeake and Delaware Canal may be cited. This canal is being lowered to sea level by the Corps of Engineers, U. S. Army. The series of continuous tidal observations made at Delaware City, Del., the present eastern terminus of the canal, and at Chesapeake City, Md., the present western end, will permit the engineer to determine very closely the expected velocity of the current through the canal and will also furnish the data necessary for the tidal mathematician to predict in advance the times of slack-water.

This paper gives a brief, non-mathematical explanation of the tides and currents from the viewpoint of the engineer, and an outline of the more important phases for those who require data only incidental to operations in connection with development projects. The paper will be confined to the procedure and methods used by the United States Coast and Geodetic Survey in its comprehensive current and tide surveys of harbors, its tidal work for the control of hydrographic surveys, and in connection with its work in the preparation of advance predictions of tides and currents. A brief description of the instruments used on this work will also be given.

The tidal work of the U. S. Coast and Geodetic Survey had its origin in the need for correcting the soundings taken in hydrographic surveys for tidal fluctuation; that is, for reducing to a common level or datum plane soundings made at varying stages of the tide. The further needs of the mariner and the engineer since the early days of the Survey have resulted in the development and extension of the work until at present it covers the following fields: Determination of datum planes for surveying and other engineering purposes; prediction of tides and currents and the preparation of annual tide and current tables; study of the mean sea level and its relation to probable crustal movements; study of methods of observing tides and currents and reducing tidal and current observations; development of instruments for observing and predicting tides; and the study of tidal phenomena in general.

While the tidal and current data obtained by the Survey in the past were primarily for the use of the mariner, the "by-products" have proven of considerable value to the engineer and to the scientist—to the engineer particularly (1) in the establishment of datum planes based on tidal definition; (2) in the defining of mean and extreme ranges of the tide at various places, a knowledge of which is necessary in marine construction; (3) in furnishing a knowledge of the current flow throughout harbors in connection with sewerage and under-harbor tunnels; (4) in rivers in connection with the economical maintenance of channels to follow natural flow; and (5) in coast erosion studies in which tides and currents, in addition to wave forms and wave impact, play a large part.*

At the beginning of tidal work investigators were most familiar with the tides of the North Atlantic, and on them the earlier tidal theories were based.

* "Problems Involved in Coast Erosion." R. S. Patton, M. Am. Soc. C. E., *The Military Engineer*, Vol. XVI, No. 90, November–December, 1924.

When observations were extended to other parts of the world, in the development of ports and the extension of maritime commerce, it was found that tides in general were not so orderly as was indicated by these comparatively simple tides of the Atlantic, the tides of the Pacific being quite different from those of the Atlantic, and the tides of the Gulf still different from those of the two oceans. This led to a study of the various types and forms of tides.

This study has also a directly practical application in the predictions of tides and the preparation of annual tide tables. Since it is physically impracticable to predict and publish the state of the tide for all ports of the world, a knowledge of the different types makes it possible to predict the times and heights of each successive high and low water for relatively few harbors as standard ports, and to refer subsidiary ports to those standard ports having tides of a similar type. In many cases it is not necessary that the secondary port be near the standard port of reference; for example, a port in the North Sea may be used as a port of reference for Nelson, B. C., Canada, a railway terminal on Hudson Bay; likewise, a constant difference exists between the time of low water at Miramichi Bay, New Brunswick, and high water in the Strait of Georgia, British Columbia, on the other side of the Continent, the two places having a similar type of tide, except that the tide curves are inverted.*

Nomenclature

As an aid in understanding the paper the nomenclature of tides and currents will be reviewed briefly:

Progressive Tidal Wave.—A wave the crest of which advances, so that the times of high and low water progress from one end of a body of water to the other.

Stationary Tidal Wave.—A wave that oscillates about an axis, high water occurring on one side of the axis at the same time that low water occurs on the other side.

Tidal Day.—Like the lunar day, the tidal day has an average length of 24 hours and 50 min.

"Moon's Meridian Passage."—Refers to the instant when the moon is directly above the meridian, and also the instant when the moon is directly below the meridian, or 180° distant in longitude. They are known as the upper and lower transits.

High Water Lunitidal Interval.—The interval between the moon's meridian passage (upper or lower) and the following high water (abbreviated H. W. I.).

Low Water Lunitidal Interval.—The interval between the moon's meridian passage and the following low water (abbreviated L. W. I.).

Tidal Current.—The horizontal backward and forward movement of the water—the current floods and ebbs.

* "The Tides and Tidal Streams with Illustrative Examples from Canadian Waters," by W. Bell Dawson, Ottawa, 1920, p. 26.

Tide.—The vertical movement of the water—the tide rises and falls.

High Water.—The maximum height reached by each rising tide (abbreviated H. W.).

Low Water.—The minimum height reached by each falling tide (abbreviated L. W.).

Range of Tide.—The difference in height between a high water and a preceding or following low water.

Mean Range.—The average difference in the heights of high and low water at any given place.

Semi-Diurnal Force.—The tide-producing force having a period of approximately one-half day. The semi-diurnal forces are the principal tide-producing forces.

Diurnal Force.—The tide-producing force having a period of approximately one day.

Long Period Forces.—The tide-producing forces having a period of one-half month, or more.

Spring Tides.—The tides occurring at the times of new moon and full moon.

Spring Range.—The range of tides at the time of spring tides.

Neap Tides.—The tides occurring at the times of the first and third quarters of the moon.

Neap Range.—The range of the tides at the time of neap tides.

Age of Phase Inequality, or Phase Age.—The phase age, generally ascribed to the effects of friction, is the lag in the response of the tide to the corresponding phases of the moon. It usually amounts to a day or two.

Perigee.—The position of the moon closest to the earth in its elliptical orbit around the earth.

Perigean Tides.—The tides with increased range occurring when the moon is in perigee.

Perigean Range.—The range of tide at the time of perigean tides.

Apogee.—The position of the moon farthest from the earth in its elliptical orbit around the earth.

Apogean Tides.—The decreased range tide occurring at the time the moon is in apogee.

Apogean Range.—The range of tide at the time of apogean tides.

Perihelion.—The position of the earth closest to the sun in its elliptical orbit around the sun.

Aphelion.—The position of the earth farthest from the sun in its elliptical orbit around the sun.

Age of Parallax Inequality or Parallax Age.—The lag in the response of the tide to the changing distance of the moon from the earth.

Declination of the Moon.—Its changing angular distance north or south of the plane of the earth's equator, as it passes through an angle of approximately 23½°, either side of the equator.

Age of Diurnal Inequality, or Diurnal Age.—The lag of the tides in response to the changing declination of the moon.

Equatorial Tides.—The tides occurring when the moon is on, or close to, the plane of the earth's equator (zero declination).

Tropic Tides.—The tides occurring when the moon in its changing declination is at its maximum semi-monthly declination and is near one of the tropics.

Synodical Month.—The month of the moon's phases, approximately 29½ days in length.

Anomalistic Month.—The month of the moon's distance, approximately 27½ days in length.

Tropic Month.—The month of the moon's declination, approximately 27⅓ days in length.

Diurnal Inequality.—The difference between morning and afternoon tides, due principally to the declination of the moon.

Higher High Water.—The higher of the two high waters of a day.

Lower High Water.—The lower of the two high waters of a day.

Lower Low Water.—The lower of the two low waters of a day.

Higher Low Water.—The higher of the two low waters of a day.

Semi-Diurnal, or Semi-Daily Type of Tide.—A tide in which two high and two low waters occur each day, but with little diurnal inequality.

Diurnal, or Daily Type Tide.—A tide in which only one high and one low water occur in a day.

Mixed Type Tide.—A tide in which two high and two low waters occur in a day, exhibiting marked diurnal inequality in the two highs or in the two lows.

Component Tide, or Component.—Each of the simple tides into which the tide of nature is resolved.

Amplitude of the Tide, or of a Component.—The semi-range of that tide or component.

The Epoch.—The phase or time of occurrence of high water reckoned from a fixed time origin.

Harmonic Analysis.—The mathematical process by which epochs and amplitudes of component tides are disentangled from actual tidal observations.

*Principal Lunar Component.**—The lunar component (M_2) which gives two high and two low waters in a tidal day of 24 hours and 50 min.

*Principal Solar Component.**—The solar component (S_2) which gives two high and two low waters in a solar day of 24 hours.

*Component N_2.**—The tidal component which takes account of the moon's perigean movement.

*Components K and O.**—The tidal components which take account of the moon's changing declination. The periods of the two components are such that at maximum declination the components are at a maximum and when the moon is on the equator they neutralize each other.

Mean Sea Level.—The plane about which the tide oscillates.

Mean Tide Level.—The plane half way between mean low water and mean high water.

* These five components are the principal ones. Between twenty and thirty components are used in accurate predictions of tides.

Plane of Mean Low Water.—The average of all low waters of a series of tide observations.

Plane of Mean Lower Low Water.—The average of all lower low waters where the tide is of the mixed type.

Plane of Mean High Water.—The average of all high waters of a series of tide observations.

Plane of Mean Lower Low Water.—The average of all lower low waters

Tidal Current.—A current brought about by tidal forces.

Non-Tidal Current.—A current brought about by causes independent of the tides, such as winds, fresh-water run-off, and differences in density and temperature.

Rectilinear Tidal Current.—The current characteristic of inland bodies of water, with slack-water and reversing direction, running flood for a period of about 6 hours, and ebb in the opposite direction for a period of about 6 hours.

Rotary Tidal Current.—The type of current in the oceans offshore. These currents do not slack and reverse, but change direction continually, passing through all points of the compass in a tidal cyle of 12 hours and 25 min.

Flood Current.—The current that sets inland or up stream.

Ebb Current.—The current that sets seaward or down stream.

Types and Forms of Tides

The tides in the various parts of the world assume a number of different types and forms, yet they may be conveniently classed as three distinct types, with their different forms. The general classification may be made as (1) semi-daily; (2) daily; and (3) mixed types of tides. Fig. 1 shows automatic

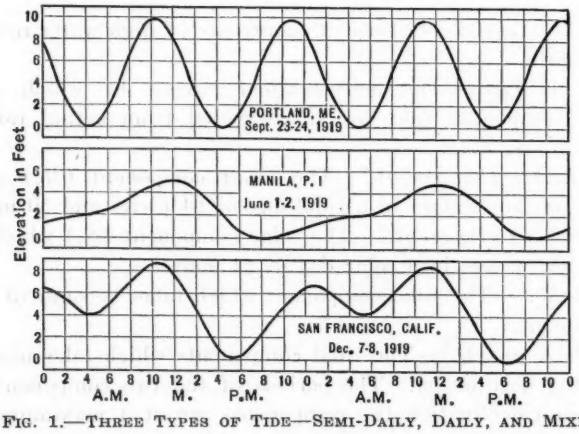


Fig. 1.—Three Types of Tide—Semi-Daily, Daily, and Mixed.

tide-gauge records, illustrating these simplest types—the semi-daily at Portland, Me., the daily at Manila, Philippine Islands, and the mixed at San Francisco, Calif. In the simple form of the semi-daily type two high and two low waters occur each day, the morning and afternoon tides being very similar. The simple form of the daily type has only one high water and one

low water during a day; while the third, the mixed type, is caused by a combination of the daily and semi-daily types. It is characteristic of this type that two high and two low waters occur each day, but the two differ both as to form and height.

Even these simplest types vary with the moon's changing declination, as illustrated in Fig. 2, reproducing tidal records at Portland and Manila. The upper graph for Portland is known as an equatorial form, which occurs when the moon is near the equator in its changing declination. The lower graph for Portland is known as the declinational form, occurring when the moon's declination is large. This variation in the height of the two high waters, or of the two low waters, during a day, is known as the diurnal inequality in the tides, and is directly related to the declination of the moon, being least when the moon is on the equator, and greatest when it is farthest north or south from the equator, causing "tropic tides". At certain times the semi-daily tide as represented by Portland approaches somewhat the mixed type, although to a minor degree in comparison with a truly mixed type.

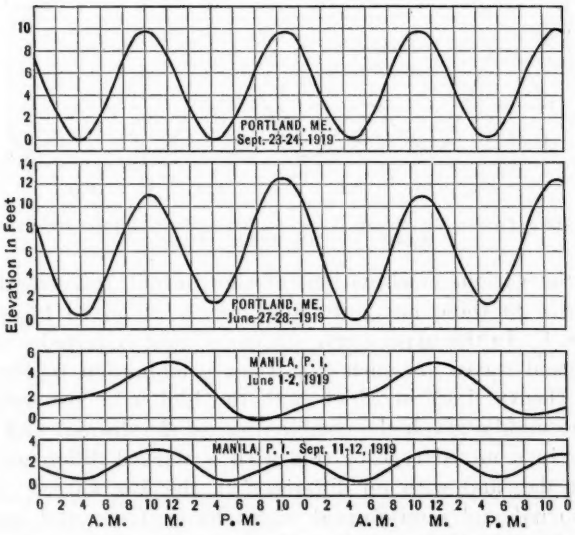


FIG. 2.—VARIATION OF TIDE CURVES DUE TO CHANGING DECLINATION OF THE MOON, EXEMPLIFIED IN TIDES AT PORTLAND, ME., AND MANILA, PHILIPPINE ISLANDS.

The diagram for Manila (June 1 and 2, 1919), illustrates a tide which ordinarily is of the daily type, but which for a few days during a lunar month at the time when the moon is nearing the equator, and, therefore, when the semi-daily forces are exerting the dominant influence, has a semi-daily variation. The remaining graph of Fig. 2 for Manila (September 11 and 12, 1919), represents the ordinary daily tide. Likewise, the tide of a port which ordinarily has a mixed type changes for a few days during the month to a semi-daily form when the moon is near the equator.

In addition to these variations two distinct forms of the mixed type occur in some localities, as illustrated by the tides at Seattle, Wash., and Honolulu, Hawaii (Fig. 3). At Seattle little variation occurs in the high waters but considerable variation, in the low waters, while at Honolulu the tide shows very little difference in the height of the low waters, but a marked difference in the height of the high waters. This difference in form is due, briefly, to different combinations of the diurnal and semi-diurnal waves—they may have the same phases and different amplitudes, different phases and the same amplitudes, or both different phases and different amplitudes. It is evident, therefore, that they may combine in various ways, giving rise to many other different forms.

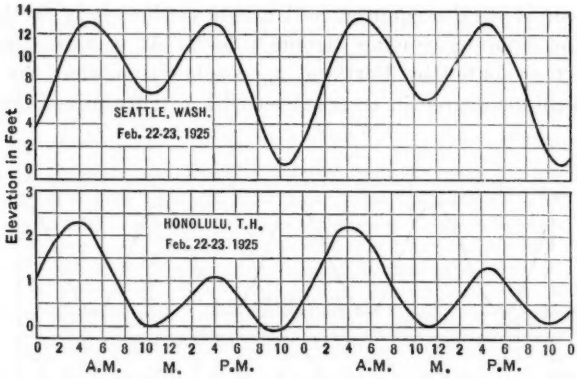


FIG. 3.—DIFFERENT FORMS OF MIXED TYPE OF TIDE EXEMPLIFIED BY TIDES AT SEATTLE, WASH., AND HONOLULU, HAWAII.

Some of the various combinations of the diurnal and semi-diurnal waves which give rise to these many different forms of mixed types of tide are shown in Fig. 4. In the upper curve the phases and amplitudes of the diurnal and semi-diurnal waves are such that their combination results in a mixed type of tide (heavy line) in which both the high and low waters of a day differ. In the middle graph the combination of the diurnal and semi-diurnal waves results in a mixed type of tide with a marked difference in the high waters of the day and with no difference in the low waters. In the lower graph the diurnal and semi-diurnal waves have phases and amplitudes the resultant of which is the "vanishing tide"; in this type the water of the falling tide remains for several hours at a fixed height at about mean sea level, then falls to the ordinary low water.

The tide in a river draining a considerable area illustrates another type of tide. The fresh-water run-off affects the tidal wave of the river to a considerable extent, causing the graph to be steep and of a comparatively short period for the flood tide and more inclined and of a longer period for the ebb tide. This is known as the "river type tide" and is illustrated by the tide at Philadelphia, Pa., and Albany, N. Y. (Fig. 5). In the upper graph the tide was rising at Philadelphia from 3:20 A. M. to 9:00 A. M. (5 hours and 40 min.), and falling from 9:00 A. M. to 4:20 P. M. (7 hours and 20 min.); in the lower

graph the tide was rising at Albany from 4:10 A. M. to 8:40 A. M. (4 hours and 30 min.), and falling from 8:40 A. M. to 4:40 P. M. (8 hours).

This type is very pronounced in a number of shallow rivers and estuaries which have considerable range of tide, and broad flats, bare or almost bare at low water. The extreme case is represented by the "bore"—the tide rises so rapidly that the meeting of the incoming tide wave with the natural fresh-water run-off of the river, coupled with the friction of the lower layers of water with the bottom, causes the incoming wave to assume the form of a

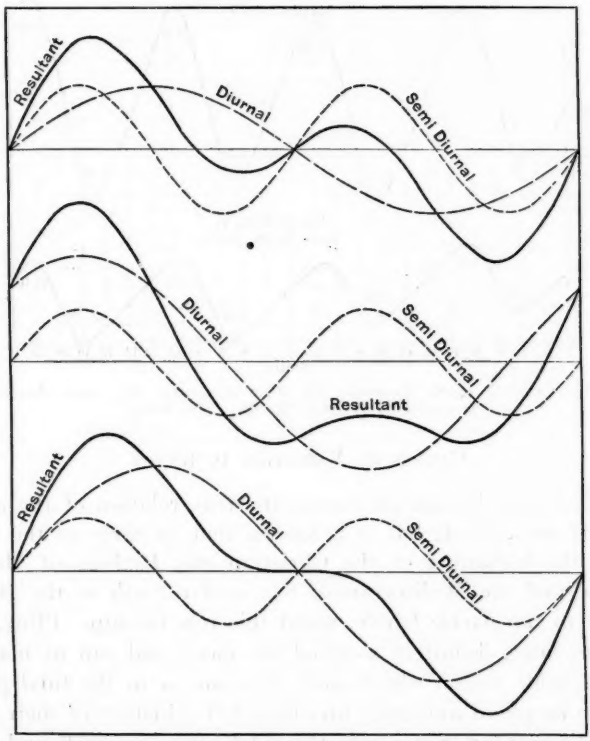


FIG. 4.—IDEALIZED TIDE CURVES ILLUSTRATING VARIOUS COMBINATIONS OF DIURNAL AND SEMI-DIURNAL WAVES WHICH GIVE RISE TO MANY DIFFERENT FORMS OF MIXED TYPES OF TIDES.

wall of water at times several feet high, which rushes up the river or estuary. Probably the most famous bore occurs in the Tsien-tang Kiang,* in China, although smaller ones are found in Turnagain Arm, Alaska, on the Severn and Wye, in England, on the Seine, in France, on the Hoogly, in India, and on the Petitcodiac, in Canada.†

As evidenced from the possible combinations illustrated in the idealized tide curves (Fig. 4), there are of course many other variations of forms of

* For description see, G. H. Darwin, "Tides and Kindred Phenomena in the Solar System," p. 63.

† Observations of the bore on the Petitcodiac River have been made at Moncton, N. B., Canada, by Dr. W. Bell Dawson, formerly Superintendent of the Tidal and Current Survey, of Canada, see "Survey of Tides and Currents in Canadian Waters," Ottawa, Ont., Canada, p. 23.

tides. The types described, however, are the principal ones. The type and form of the tides are considerably modified at times by the changing declination of the moon; that is, its position relative to the equator, as well as by terrestrial causes. There are also periodic variations in the range of the tide; that is, the heights to which the tide rises at high water and to which it falls at low water. These variations in range are brought about by astronomical causes.

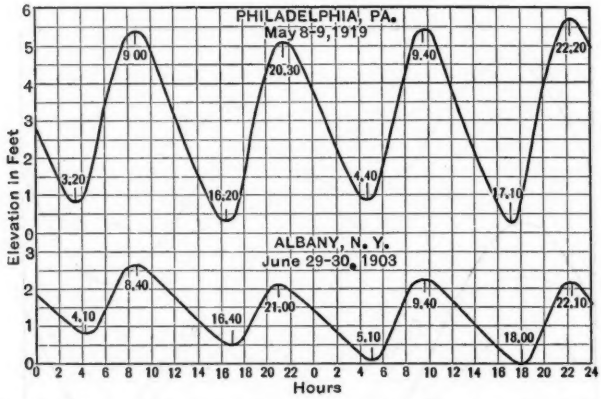


FIG. 5.—TIDE CURVES FROM RECORDS AT PHILADELPHIA, PA., AND ALBANY, N. Y., ILLUSTRATING THE "RIVER TYPE TIDE."

CAUSES OF VARIATION IN RANGE

History does not disclose just when the true relation of the moon to the tides was first recognized; but it is known that as early as the Third Century, before the beginning of the Christian era, Pytheas of Massilia, who had sailed beyond the Mediterranean Sea as far north as the British Isles, and probably to the Arctic Circle, noted this relationship. Pliny, the Elder, four centuries later, definitely ascribed the moon and sun as makers of the tide. The probable reason why history is vague as to the tidal phenomenon is because the people of antiquity who have left a history of their times lived near the Mediterranean Sea where the tidal range is small and its regular fluctuations are frequently modified and at times completely masked by meteorological conditions.

In the sixteen centuries following Pliny, the Elder, no progress was made toward a solution of the problem of the tides. The discovery of the law of gravitation by Newton (toward the end of the Seventeenth Century) furnished a rational explanation of the forces exerted by sun and moon. Newton showed that the tide was a natural consequence of the law of gravitation; having proved this he did not carry his investigations further, but left the development of the different theories to the scientists following him.

It is now well known, of course, that the tides are caused by the attraction of sun and moon on the rotating earth, the moon being the principal agent, since it is closer to the earth and since the tidal forces exerted by a foreign

body vary directly as its mass and inversely as the cube of its distance from the earth. It is evident, therefore, that any variations in the distance from the earth of these two tide-producing bodies, or any variations in their relative positions with reference to the earth, will bring about corresponding variations in the tidal forces exerted and thus a periodic variation in the range of tide.

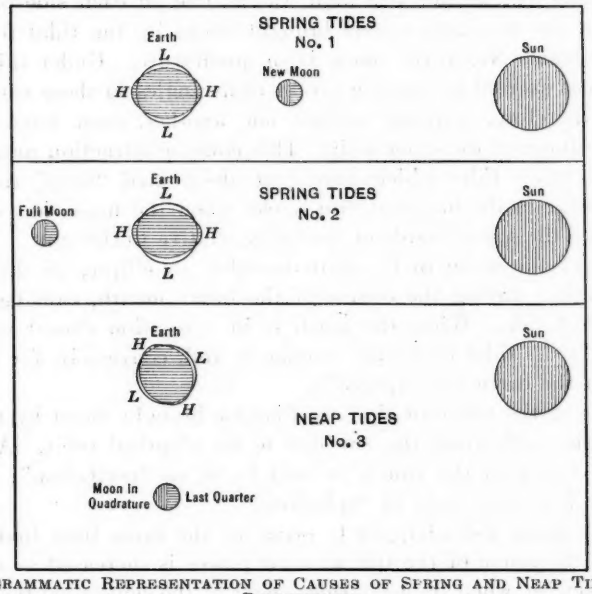


FIG. 6.—DIAGRAMMATIC REPRESENTATION OF CAUSES OF SPRING AND NEAP TIDES AND THEIR DIFFERENCES.

A non-mathematical illustration of these varying astronomical conditions is furnished by Fig. 6 which assumes the earth to be covered with water to a uniform depth. When the moon is over any point its attraction for the water under it is greater than its attraction for the solid earth; the tendency is, therefore, to pull the water away from the earth and thus cause high water. Likewise, the moon's attraction for the solid earth is greater than its attraction for the water on the opposite side of the earth from the moon; its tendency then is to draw the earth away from the water and make high water on the opposite side. The moon, therefore, as it travels around the earth causes in general two high waters each day at a place, one at the upper, and one at the lower, transit, with two low waters in between. While these conditions are not exactly true in Nature, Fig. 6 will serve to illustrate diagrammatically the causes of the so-called "spring" and "neap" tide and their differences. This diagram is untrue to Nature in that high water does not actually follow directly under the moon but has a lag, due to viscosity of water, to friction, and to land masses; and, in addition, that the tidal forces exerted by the moon and sun are tangential and not vertical, that is, the direct lifting power of the moon and sun is slight, and the water rather is drawn together from the two sides of the earth under these bodies.

In Diagrams No. 1 (Fig. 6), the moon and sun are in line relative to the earth, and their tidal forces acting through gravity on the earth and oceans are in concert and bring about the large tides which have been designated "spring" tides. In Diagrams No. 2, although the moon is on the opposite side of the earth from the sun, the tidal forces of both bodies are again in unison and produce the spring tides which occurred a fortnight before at the time of new moon, since the moon's two high waters, one on each side of the earth, combine with the two high waters brought about by the tidal forces of the sun. In Diagrams No. 3, the moon is in quadrature. Under this condition the tidal forces exerted by the sun are at right angles to those exerted by the moon, and, therefore, working against one another, each force tending to minimize the force of the other body. This counter-attraction naturally gives rise to small range tides which have been designated "neap" tides. These neap tides occur again in about two weeks when the moon has come to its first quarter on the opposite side of the earth relative to the sun.

The path of the moon in its orbit describes an ellipse, its distance from the earth varying during the course of the lunar month, causing variations in the range of tide. When the moon is in a position closest to the earth (in "perigree"), its tidal forces are strongest; and, conversely, for its position farthest from the earth (in "apogee").

Variations in the range of tide are likewise brought about by the varying distance of the earth from the sun due to its elliptical orbit. At the time the earth is closest to the sun it is said to be in "perihelion"; when it is farthest away it is said to be in "aphelion".

When the spring tides happen to occur at the same time that the moon is in perigee, the range of the tide at some places is increased as much as 40 per cent. Likewise, when the neap tides occur at the same time that the moon is in apogee, the range is less than the mean by about 40 per cent.

An example from Fort Hamilton, New York Harbor, will serve to illustrate these differences.* There the spring, neap, apogean, perigean, and mean ranges are as follows:

Mean	4.7 ft.
Spring	5.6 ft., or 20% greater than the mean range.
Neap	3.8 ft., or 20% less than the mean range.
Perigean	5.8 ft., or 23% greater than the mean range.
Apogean	3.8 ft., or 20% less than the mean range.

The moon was full and also in perigee on July 15, 1916. On July 16, a day later, the tide record from Fort Hamilton shows a range of 6.5 ft., or 39% greater than the average range. This lag of one day in the response of the tide to the tide-producing forces is known as the "age of the tide" and is generally ascribed to the effects of friction.

The moon was in its third quarter and also in apogee on October 16, 1916. The following day the range at Fort Hamilton was 3.2 ft., or 32% less than the mean range.

* "Flood and Ebb in New York Harbor," by H. A. Marmer, *Geographic Review*, Vol. XIII, July, 1923, p. 420.

This subject of types, forms, variations in tidal range, and many other problems have required study and solution by tidal mathematicians before tide predictions could be presented to the engineer and to the mariner in their present accurate state. A study of the different types of tides and the behavior of the tide in different parts of the world played a large part in the conception and in the development of the latest tidal theory—the stationary wave theory of R. A. Harris, a mathematician of the U. S. Coast and Geodetic Survey.

Tidal Theories

Several tidal theories, based on the law of gravitation and Newton's theory, have been advanced since his time,* each looking toward a finally accepted solution of the tidal problem, among them Bernouilli's Equilibrium, or Static, Theory; Airy's Canal Theory; Whewell's Progressive Wave, or Southern Ocean Theory; and, finally, the latest—the Stationary Wave Theory of Harris. Harris' theory is opposed to the older ones. For the idea advanced by previous theories, namely, a single world phenomenon, it substitutes regional oscillatory areas as the origin of the dominant tides of the various oceans, these oscillations being set up and maintained by the periodic tidal forces of the moon and sun.

Clearly, a wave in continuous motion does not imply that the water is moving in the same direction—it is only the forward motion of a shape as evidenced by the kind of wave traveling in from the ocean toward the land. A somewhat similar motion occurs in a rope held in the hand and violently agitated, or a wave in a pond caused by the dropping of a pebble in the water. In the case of the rope a wave will travel its length, the rope itself remaining fixed in so far as progression in direction of travel of the waves. Such a wave is known as a progressive wave (illustrated by the lower diagram, Fig. 7); the older tidal theories depended on the progressive wave as an explanation of the origin of the dominant tides of the ocean. An altogether different kind of wave may travel through a body of water. The upper diagram in Fig. 7 represents a vessel partly filled with water. If one end is raised and immediately lowered, a wave will be started throughout the vessel, and the water put into oscillation, causing a greater depth at one end than at the other, that is, high water at H' and low water at L', with a nodal line, or no change of level, at N. As the oscillation is continued, high water occurs at H and low water at L, and again a nodal line at N. High water will occur at one end when it is low water at the other, and high or low water will occur simultaneously throughout each half of the vessel. This kind of wave is known as the stationary wave.

The period of this wave or oscillation depends on the length of the vessel and the depth of the water. To cause this oscillation to continue it is only necessary to continue the disturbing force. If this force be applied at regular intervals coinciding with the period of the vessel, the maximum effect

* For general description of the different theories see *Scientific Monthly*, Vol. XIV, No. 3, March, 1922, H. A. Marmer, pp. 209–218; for detailed explanation, see "Manual of Tides," by Harris, Pt. 1, pp. 386–465.

is obtained. Similar oscillations occur in most lakes in which the whole body of water swings back and forth with a period of from 10 min. to 1 hour and with a range of several inches. These oscillations, known as seiches, are caused by sudden changes in atmospheric pressure or by winds over the lakes. Studies of seiches have been made for a number of lakes and given in detail in various publications.*

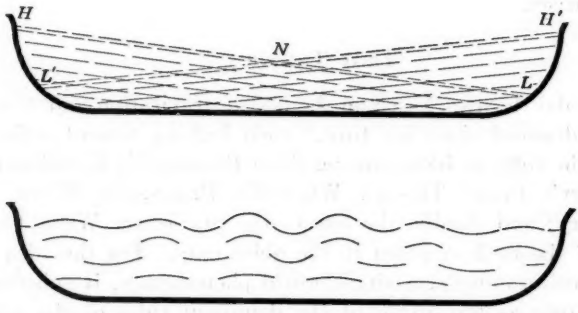


FIG. 7.—GRAPHIC ILLUSTRATION OF STATIONARY AND PROGRESSIVE TIDE WAVES. UPPER DIAGRAM IS BASIS OF A STATIONARY THEORY OF TIDES.

In brief, therefore, such a system of waves forms the basis of Harris' stationary wave theory—that the dominant tides of the world are the results of stationary waves, these oscillations being set up and maintained in the various parts of the oceans by the periodic tidal forces of moon and sun.

In most bodies of inland water connected with the ocean the tidal wave is a progressive wave thrown out by a near-by oscillatory wave in the ocean. The rate of advance of such a progressive wave up an inland body of water, such as Chesapeake Bay, is expressed approximately by the formula, $r = \sqrt{g\,h}$, in which, r is the rate of advance; g, the acceleration due to gravity which is about 32.2 ft. per sec.; and h, the average depth of the water.

If the size and depth of a body of water is such as to maintain an independent stationary wave, its tides may be caused by its own stationary wave; or again in some bodies of water the tide may be caused partly by a progressive wave and partly by a stationary wave, as, for example, in Long Island Sound and in the Bay of Fundy. The period of a body of water having a stationary wave is expressed in the formula, $T = \dfrac{4\,L}{\sqrt{g\,h}}$, in which, L is the length of the body in the direction of oscillation. In such a body high (and also low) water occurs almost simultaneously throughout its length and the range of the tide increases toward and is greatest at the head; in a body with a progressive wave high and low water advance at a rate dependent on the depth, and the range of tide generally decreases toward the head. The tides in the Bay of Fundy are the best example of the wave of the stationary type.

* "The Tides and Kindred Phenomena in the Solar System," by G. H. Darwin, p. 21, and "Effects of Winds and Barometric Pressures on the Great Lakes," by the late John F. Hayford, M. Am. Soc. C. E., Carnegie Inst. of Washington, 1922.

The Bay of Fundy may be considered as a part of the vessel of water shown in the upper diagram, Fig. 7, the tide occurring almost simultaneously throughout the length of the Bay, the range at the mouth being from 6 to 10 ft., and increasing up the Bay to about 20 ft., and, finally, at the head of the Bay to more than 40 ft. At Nantucket, off the coast of Massachusetts, the range of tide is 3.3 ft., at Monomoy Point, Mass., 3.7 ft., at Nauset, Mass., 6.0 ft., increasing gradually to 20.9 ft. at St. Johns, N. B., and to 41.2 ft. at Moncton, N. B.

The stationary wave theory, then, would have the tides in the various oceans distinct problems. They may be due to a progressive wave thrown out by a near-by oscillatory area of the open ocean, or if the size and depth of the body is such as to maintain an independent stationary wave, its tides may be caused by its own stationary wave; or, again, in some bodies of water, the tide may be caused partly by a progressive wave and partly by a stationary wave. According to the stationary wave theory the configuration of the bottom, the size of the body of water, and the relation of the period of the tidal forces to the period of the body of water make the tide more a local phenomenon.

While from astronomical considerations a mathematical expression for the tide-producing forces may be derived, from which a theoretical tide may be obtained, the actual tides of Nature are so modified by land masses that they have little or no resemblance to these theoretical tides. For the prediction of tides at any place, therefore, it is necessary first to make actual observations, extending over a period of a month to a year or more. From these observations constants are derived which may be introduced into the theoretical expressions for predicting tides as they actually occur in Nature.

Prediction of Tides

In the days of shallow-draft vessels advance knowledge of the state of the tides was of little importance, but as the draft of vessels approached the depth of water over bars and in harbors, such knowledge became of importance to the navigator. It has also become of value to the engineer in the placing of bridge spans, and in a number of similar engineering operations; to watering places for the convenience of the bather; to the fisherman in planning his fishing trips; and to a number of diverse interests too numerous to mention. Figs. 8 and 9, showing high and low tides at Anchorage, Alaska, forcibly impress the need of tide predictions in places having a considerable range of tide and comparatively shallow water at the time of low tide.

The first tide table of record is in the Library of the British Museum. It is a manuscript written in the Thirteenth Century, and gives the time of the tide at London Bridge relative to the age of the moon and not for calendar days. Each of the leading maritime nations publishes tide tables, and since the Coast and Geodetic Survey is the Bureau of the United States Government charged with the furnishing of information to the mariner, the important function of predicting and issuing tide tables naturally devolves on this Service. Tide tables were first published by the Survey in 1853. For the first fourteen years they consisted of a somewhat elaborated means for

enabling the mariner to make his own predictions as occasion arose. In 1867 tables were issued for a few ports on each coast of the United States. They have since been considerably increased in size and scope until now they cover the entire maritime world.

While approximate predictions of the time of high water for a particular day can be made by adding 50 min. for each day since the full and change of the moon to the lunitidal interval (known as the Establishment of the Port), so many factors are involved in the prediction of tides that for accurate results a tide-predicting machine is necessary.

Prior to 1885 the predictions of the tides were computed by means of empirical tables and graphs. Beginning with that year the predictions were made by use of an ingenious machine* devised by Mr. William Ferrel, of the U. S. Coast and Geodetic Survey. This machine differed somewhat from that of Lord Kelvin, who conceived the idea of such a machine and who devised the first one in 1872. When predictions were extended, however, by the Survey to cover the world, it was found that the Ferrel machine was not well adapted to the prediction of certain types of tides, and since 1910 the predictions have been made on a new machine, also designed and constructed in the office of the Survey. This machine is shown in Fig. 10.† By its use a tide table for any single port can be made in about 10 hours, giving the time and height of every high and low water for every day of the year.

For the purpose of predictions the tide is assumed to be composed of a number of elementary constituent tides, each due to the motions of the sun and of the moon and to their relative positions during the year; and, since the tides are local phenomena, the first step in the making of a tide table for any port is to obtain continuous tidal observations at that port. These observations are then subjected to the harmonic analysis which resolves the tide into its constituent simple harmonic waves, or harmonic constants, the periods of which have astronomical definitions, while the amplitudes and phases are derived from actual observations of the tide.

Before beginning the predictions for any port the amplitudes (semi-ranges), and the phases (times of occurrence of high water reckoned from a fixed origin), as obtained from this analysis for each constant, are set on the proper cranks and dials of the machine. These cranks and dials are arranged so that when the machine is put in motion by a handle and gears, it will sum up the effects of all these elementary waves into a single wave representing the tide of Nature. Further, the machine is so constructed that this summation is indicated on a large dial (Fig. 10) as high and low water, and also the time of occurrence is given in standard civil time. The U. S. Coast and Geodetic Survey machine will take account of thirty-seven of these elementary tides.

It is obviously impracticable to predict the tides for all ports. To obtain the state of the tide at a place where no full predictions are made, it is referred to some other port having a similar type of tide for which predictions have been made as a standard port. The Survey makes full predictions for eighty-

* Appendix No. 10, U. S. Coast and Geodetic Survey Report, 1883, p. 253.

† "Description of the United States Coast and Geodetic Survey Tide Predicting Machine No. 2" pub. by the U. S. Coast and Geodetic Survey.

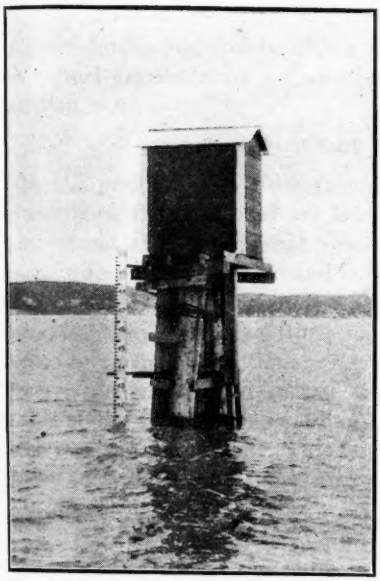


FIG. 8.—HIGH WATER AT TIDE STATION, ANCHORAGE, ALASKA, READING 41.3 FEET.



FIG. 9.—LOW WATER AT TIDE STATION, ANCHORAGE, ALASKA, READING 7.5 FEET.

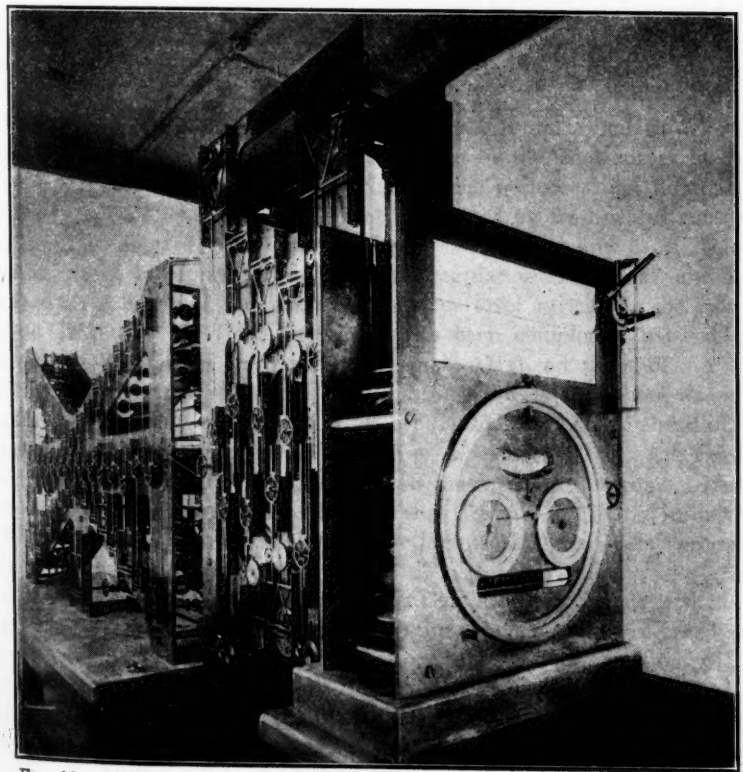


FIG. 10.—FRONT VIEW OF TIDE-PREDICTING MACHINE OF THE U. S. COAST AND GEODETIC SURVEY.

three sta
be obtai
maritim

In th
for any
place, e
hourly c
tions ar
observat
average
between
heights
which r
subject
small r

In s
compar
wind,
and in
Bay, f
tides t
heavy
predic

Th
data i
from
the lo
water
fluctu
decre
until
fresh
cond
perio
fluct
up t
pred
exis
con

as
obs
bee
the
Fe



three standard ports, and furnishes a table of data from which predictions may be obtained expeditiously for about 3 500 subsidiary ports, covering the entire maritime world.

Accuracy of Tide Tables

In the prediction of tides (also for mean ranges and lunitidal intervals) for any port, the data used are based on the longest series obtainable at that place, except that, in general, due to the labor involved in the summing of hourly ordinates of the tide and in the harmonic analysis, one year's observations are considered sufficient for all practical purposes. The average of these observations will give the times and heights of the astronomical tide under average or normal conditions. Exact agreement, however, cannot be expected between predictions and actual times of occurrence, since the times and heights of the tide will be modified by the prevailing meteorological conditions, which may not be normal. This is particularly true of upper reaches in rivers subject to freshet conditions, and also of comparatively shallow bays with a small range of tide.

In some bodies of water in which the range of tide is small and the depth comparatively shallow, the water will be banked up or blown out by a strong wind, causing considerable differences between predictions and actual tides, and in extreme cases completely masking the normal tide. In Chesapeake Bay, for example, a stiff northwest wind blowing for a few days will cause the tides to be later and the heights lower than those predicted. Then, again, a heavy northeast storm piles up the water in the Bay causing tides higher than predicted.

The necessity for long series of observations to obtain final values in tidal data in a river subject to freshets is brought out by the tidal curves in Fig. 11, from actual observations at Albany, N. Y. The periodic tidal fluctuation in the lower graph (May 27) is only slightly modified by fresh-water run-off, low water reading about 2½ ft. on the staff and the range being 3 ft. The tidal fluctuations on the upper graph, although regular on March 27 and 28, are decreased in range and the whole mean river level raised on the tide staff, until on March 29 the tidal fluctuation has been completely masked by the freshet, and the level of the river has risen to 11 ft. on the staff. A similar condition is illustrated on the middle graph (April 11) when there was a slight periodic tidal fluctuation with the river level raised to 11 ft. on the staff. This fluctuation is completely masked by 8:00 P. M., April 12, with the river level up to 18 ft. on the staff. These conditions, however, are extreme. Most tide predictions are made for bodies of water that are not so greatly influenced by existing meteorological conditions, either because of large tidal range or of considerable depth of water.

As an illustration of the close agreement of these predictions with the tides as they actually occur at stations not influenced to such a marked degree, observations and predictions at San Diego, Calif., and Fernandina, Fla., have been compared for the months of March and August, 1923. At San Diego, the tide is of the mixed type and has a range varying from 2 to 6 ft.; at Fernandina, the tide is of the semi-daily type and has a range of 4 to 9 ft.

At San Diego for the whole month of March, 1923, 60% of the predicted times differed by not more than 0.1 hour from the observed, and 87% by not more than 0.2 hour. For August, at this station, 48% of the predicted times differed from the observed by not more than 0.1 hour, and 75% by not more than 0.2 hour. The greatest difference in each month between the predicted and observed times of tides was 0.8 hour, or about 50 min.

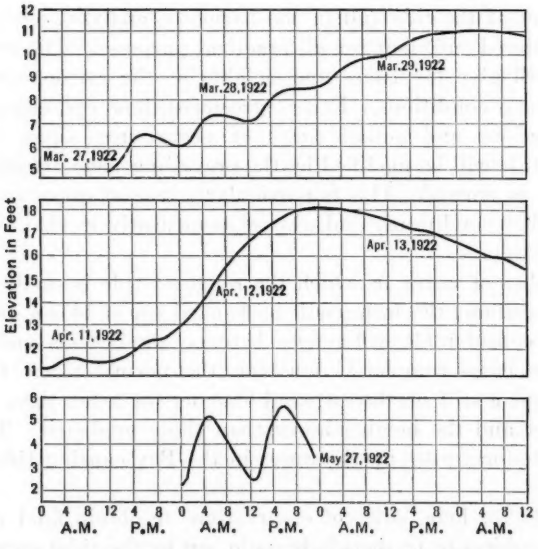


FIG. 11.—TIDE CURVES FROM RECORD AT ALBANY, N. Y., ILLUSTRATING RIVER TYPE TIDE AND EFFECTS OF FRESHETS IN UPPER REACHES OF RIVER.

At Fernandina for the whole of the month of March the percentages are as follows: 36% of the predicted times differed by not more than 0.1 hour from the observed tide; 59% by not more than 0.2 hour; and 76% by not more than 0.3 hour. During the month of August, 39% differed by not more than 0.1 hour from the observed; 55% by not more than 0.2 hour; and 79% by not more than 0.3 hour. During the month of March the greatest difference was 0.7 hour, or about 40 min., and for August the greatest difference was 0.6 hour, or about 35 min. It should be borne in mind, however, that no corrections were applied to the predicted times of tide for meteorological conditions; and, further, that near the times of high and low water the height of the tide is changing slowly and, therefore, it is not practical to determine the time of the observed high or low water with a precision greater than 0.1 hour.

It is quite evident, therefore, why a continuous record at different places of the rise and fall of the water, brought about by the tides, early became of importance to modern life. Not only were these data necessary for commerce in the advance predictions of the future state of the tides in harbors and for the reductions of soundings in hydrographic surveys, but they were of import-

ance to the theoretical study of the tides in general. These records are also of considerable value to the engineer for defining datum planes; to the geodesist in connection with his level nets and the reduction of his large-scheme triangulation systems to mean sea level; and to the geologist for testing the stability of coast lines by reference to accurately determined mean sea level.

Use of Tide Tables

For those who may not be familiar with the tide tables and their use the following examples, together with the facsimiles of pages* of tide tables (Figs. 12 and 13), will illustrate the facility with which the predicted times of high and low waters may be obtained for any port listed in the tables, as a standard port, or as a subsidiary port in Table 2 of the Tide Tables (see Fig. 13).

As an example: What is the predicted time and height of high water at New York (Governor's Island) on the afternoon of December 25, 1926? Referring to Fig. 12, opposite December 25, in the double column, "High", it is found that the afternoon high water occurs at 12:20 P. M., and that the height is 3.5 ft. above the datum of soundings on U. S. Coast and Geodetic Survey charts.

If predictions are desired for a subsidiary port for which full predictions are not listed among the standard ports, Table 2 of the Tide Tables (Fig. 13), furnishes the necessary data.

As an example: What is the predicted time and height of high water at West Point, N. Y., on the afternoon of December 25, 1926?

The time of high water just found for New York, the standard port under which West Point is listed in Table 2 of the Tide Tables, is used to obtain that for West Point as follows (Fig. 13):

	Time.	Height.
High water at New York.....	12:20 P. M.	3.5 ft.
Tidal difference .....+	3:25 P. M.	—1.8 ft.
High water at West Point.....	15:45, or 3:45 P. M.	1.7 ft.

It might be well to explain briefly the remainder of columns in Table 2 of the Tide Tables (Fig. 13). It is evident that the first and second narrow columns are the latitudes and longitudes of the ports, and the third and fourth narrow columns, the time and height differences to be applied to the standard port predictions to obtain those for minor ports. In Fig. 13, the column, "Ratio of Ranges" gives the ratio of the rise and fall of the tides at the standard and at the secondary ports. For example, the mean rise and fall of the tide at West Point is 0.6 of that at New York. The column headed "High Water Interval", gives for West Point 11 hours and 30 min., which means that high water occurs there on an average 11 hours and 30 min. after the moon crosses the meridian of West Point. The last two columns refer to the ranges, the "mean range" being the mean of all the ranges of the tide, and the "spring range", the mean range of the spring tides.

* From "Atlantic Coast Tide Tables" for 1926.

NEW YORK (Governors Island), NEW YORK, 1926.

Tide Tables.

Day.	OCTOBER.				Day.	NOVEMBER.				Day.	DECEMBER.			
	High.		Low.			High.		Low.			High.		Low.	
	Time.	Ht.	Time.	Ht.		Time.	Ht.	Time.	Ht.		Time.	Ht.	Time.	Ht.
	h. m.	ft.	h. m.	ft.		h. m.	ft.	h. m.	ft.		h. m.	ft.	h. m.	ft.
1	4 17	3.8	10 24	0.7	1	5 17	4.2	11 31	0.4	1	5 15	4.1	11 43	0.1
	4 34	4.4	10 58	0.4		5 28	4.1	11 49	0.0		5 25	3.7	11 48	−0.2
2	5 07	4.0	11 14	0.6	2	5 55	4.4	. . .	. . .	2	5 52	4.4	. . .	. . .
	5 21	4.4	11 44	0.3		6 05	4.2	12 13	0.2		6 05	3.7	12 26	−0.2
3	5 50	4.2	11 59	0.5	3	6 30	4.6	0 27	0.0	3	6 26	4.6	0 26	−0.3
	6 03	4.5	. . .	. . .		6 40	4.2	12 54	0.1		6 43	3.8	1 06	−0.3
4	6 30	4.3	0 25	0.1	4	7 01	4.7	1 03	−0.1	4	7 01	4.8	1 02	−0.3
	6 41	4.5	12 43	0.3		7 14	4.2	1 33	−0.1		7 21	3.8	1 47	−0.5
5	7 06	4.5	1 05	0.1	5	7 31	4.9	1 37	−0.1	5	7 36	4.9	1 39	−0.4
	7 16	4.6	1 23	0.2		7 47	4.2	2 11	−0.1		8 00	3.8	2 27	−0.6
6	7 38	4.7	1 41	0.0	6	8 02	5.0	2 09	−0.1	6	8 14	4.9	2 17	−0.3
	7 48	4.6	2 01	0.2		8 20	4.1	2 49	−0.2		8 41	3.8	3 10	−0.6
7	8 08	4.8	2 15	0.0	7	8 36	5.0	2 42	0.0	7	8 55	4.9	2 59	−0.3
	8 18	4.5	2 39	0.1		8 56	4.0	3 28	−0.1		9 28	3.8	3 55	−0.6
8	8 36	4.9	2 47	0.1	8	9 11	5.0	3 16	0.1	8	9 40	4.8	3 45	−0.2
	8 48	4.4	3 15	0.1		9 37	3.9	4 11	−0.1		10 19	3.7	4 44	−0.6
9	9 04	4.9	3 18	0.2	9	9 53	4.9	3 56	0.2	9	10 31	4.6	4 38	−0.1
	9 20	4.3	3 53	0.2		10 25	3.8	4 58	0.0		11 17	3.7	5 39	−0.5
10	9 37	4.9	3 49	0.3	10	10 42	4.7	4 44	0.4	10	11 28	4.4	5 40	0.1
	9 58	4.2	4 31	0.3		11 20	3.7	5 53	0.1		. . .	. . .	6 36	−0.5
11	10 16	4.8	4 22	0.4	11	11 37	4.6	5 45	0.5	11	0 24	3.7	6 51	0.1
	10 41	4.0	5 15	0.4		. . .	. . .	6 55	0.1		12 32	4.2	7 37	−0.5
12	11 02	4.8	5 03	0.5	12	0 27	3.7	7 00	0.5	12	1 35	3.8	8 03	0.1
	11 31	3.9	6 07	0.5		12 43	4.4	7 59	0.0		1 45	4.0	8 38	−0.6
13	11 55	4.7	6 00	0.7	13	1 45	3.8	8 17	0.5	13	2 46	4.0	9 13	−0.1
	. . .	. . .	7 10	0.5		1 59	4.4	9 01	−0.2		2 58	4.0	9 36	−0.7
14	0 32	3.8	7 06	0.7	14	2 59	4.1	9 26	0.2	14	3 48	4.3	10 14	−0.3
	12 58	4.6	8 18	0.4		3 13	4.4	9 59	−0.4		4 05	3.9	10 30	−0.8
15	1 46	3.8	8 26	0.7	15	4 04	4.4	10 29	−0.1	15	4 45	4.6	11 12	−0.6
	2 12	4.6	9 23	0.2		4 20	4.5	10 53	−0.6		5 05	4.0	11 23	−0.9
16	3 07	4.1	9 38	0.5	16	5 02	4.8	11 27	−0.4	16	5 37	4.9	. . .	. . .
	3 29	4.7	10 22	−0.1		5 20	4.6	11 45	−0.8		6 01	4.0	12 06	−0.8
17	4 18	4.4	10 42	0.1	17	5 55	5.2	. . .	. . .	17	6 28	5.0	0 14	−1.0
	4 38	4.9	11 17	−0.4		6 16	4.7	12 21	−0.7		6 53	4.0	12 57	−1.0
18	5 18	4.8	11 42	−0.2	18	6 45	5.4	0 35	−0.9	18	7 16	5.1	1 03	−0.9
	5 39	5.1	. . .	. . .		7 09	4.7	1 13	−0.9		7 43	4.0	1 46	−1.1
19	6 13	5.2	0 10	−0.7	19	7 34	5.5	1 23	−0.9	19	8 02	5.0	1 50	−0.9
	6 34	5.2	12 37	−0.5		7 59	4.6	2 03	−1.0		8 31	3.9	2 34	−1.1
20	7 05	5.5	0 59	−0.8	20	8 21	5.5	2 11	−0.8	20	8 45	4.9	2 37	−0.7
	7 27	5.2	1 30	−0.7		8 49	4.4	2 53	−0.9		9 16	3.8	3 20	−1.0
21	7 55	5.7	1 48	−0.9	21	9 07	5.3	2 59	−0.6	21	9 28	4.7	3 24	−0.5
	8 18	5.1	2 21	−0.8		9 39	4.3	3 43	−0.8		10 04	3.6	4 07	−0.8
22	8 43	5.7	2 36	−0.8	22	9 55	5.0	3 49	−0.4	22	10 10	4.4	4 12	−0.3
	9 09	4.9	3 14	−0.8		10 30	3.9	4 34	−0.6		10 50	3.5	4 53	−0.6
23	9 32	5.5	3 25	−0.6	23	10 43	4.6	4 40	0.0	23	10 53	4.1	5 00	0.0
	10 01	4.6	4 05	−0.6		11 24	3.7	5 25	−0.4		11 38	3.3	5 41	−0.5
24	10 22	5.3	4 16	−0.3	24	11 33	4.3	5 34	0.3	24	11 36	3.8	5 51	0.2
	10 55	4.3	4 58	−0.4		. . .	. . .	6 17	−0.2		. . .	. . .	6 29	−0.3
25	11 15	4.9	5 08	0.0	25	0 19	3.5	6 30	0.5	25	0 28	3.3	6 45	0.4
	11 52	4.0	5 53	−0.1		12 26	4.0	7 11	0.0		12 20	3.5	7 18	−0.1
26	. . .	. . .	6 04	0.4	26	1 17	3.4	7 28	0.7	26	1 19	3.2	7 42	0.5
	12 10	4.6	6 49	0.1		1 21	3.8	8 05	0.1		1 09	3.3	8 07	−0.1
27	0 52	3.8	7 04	0.6	27	2 13	3.4	8 26	0.7	27	2 11	3.3	8 38	0.5
	1 08	4.3	7 47	0.3		2 17	3.6	8 55	0.1		2 03	3.2	8 55	0.0
28	1 54	3.7	8 02	0.8	28	3 06	3.5	9 21	0.6	28	3 01	3.4	9 32	0.4
	2 09	4.1	8 42	0.4		3 09	3.6	9 43	0.1		2 58	3.1	9 41	−0.1
29	2 53	3.7	9 01	0.8	29	3 54	3.7	10 12	0.5	29	3 49	3.6	10 23	0.2
	3 05	4.0	9 35	0.3		3 58	3.6	10 28	0.0		3 53	3.1	10 27	−0.1
30	3 47	3.8	9 56	0.7	30	4 37	3.9	10 59	0.3	30	4 32	3.9	11 11	0.0
	3 57	4.0	10 23	0.3		4 43	3.6	11 06	−0.1		4 44	3.2	11 11	−0.2
31	4 35	4.0	10 45	0.6						31	5 14	4.1	11 57	−0.3
	4 44	4.1	11 06	0.1							5 31	3.3	11 52	−0.4

Time meridian, 75° W. Heavy-faced type indicates p. m. tides. Heights are reckoned from mean low water, the datum of soundings on Coast and Geodetic Survey charts.

FIG. 12.—REPRODUCTION OF PART OF TABLE 1, SHOWING DATA INCLUDED IN TIDE TABLES FOR PORT OF NEW YORK.

TABLE 2.—UNITED STATES, Atlantic Coast.

New York to New Jersey.	Latitude.	Longitude.	Tidal difference. Time of tide.	Tidal difference. Height of H. W.	Ratio of ranges.	High water interval.	Mean range of tide.	Spring range of tide.
	° ′ North.	° ′ West.	h. m.	feet.		h. m.	feet.	feet.
New York, *Staten Island*—Continued.	Standard port, New York, page 46.						Time meridian, 75° W.	
Port Richmond, Kill van Kull	40 38	74 09	+0 20	+0.1	1.0	8 11	4.5	5.4
New Brighton, Kill van Kull	40 39	74 06	−0 15	+0.1	1.0	7 56	4.5	5.4
St. George	40 39	74 04	−0 15	0.0	1.0	7 53	4.4	5.3
Fort Wadsworth Light, The Narrows	40 36	74 03	−0 25	+0.2	1.0	7 40	4.6	5.6
New York, *New York Harbor.*								
Bath, Gravesend Bay	40 36	74 00	−0 30	+0.4	1.1	7 35	4.8	5.7
Fort Hamilton, The Narrows	40 37	74 02	−0 25	+0.3	1.1	7 40	4.7	5.7
Bay Ridge	40 38	74 02	−0 15	+0.1	1.0	7 49	4.5	5.4
Gowanus Bay	40 40	74 01	−0 10	0.0	1.0	7 56	4.4	5.3
New York, Governors Island	40 42	74 01	0 00	0.0	1.0	8 04	4.4	5.4
Hudson River, New York and New Jersey.								
New York, The Battery	40 42	74 01	0 00	0.0	1.0	8 06	4.4	5.3
Jersey City, Penn. R. R. Ferry, N. J.	40 43	74 02	+0 05	−0.1	1.0	8 09	4.3	5.3
New York, Desbrosses Street	40 43	74 01	+0 05	−0.1	1.0	8 10	4.3	5.2
Hoboken, 14th Street, N. J	40 45	74 02	+0 10	−0.2	1.0	8 16	4.2	5.2
Weehawken, Day Point, N. J	40 46	74 01	+0 20	−0.2	1.0	8 23	4.2	5.1
New York, W. 72d Street	40 47	73 59	+0 30	−0.3	0.9	8 35	4.1	5.0
General Grant's Tomb, 122d Street	40 49	73 58	+0 40	−0.4	0.9	8 45	4.0	4.9
Manhattan Iron Works, 143d Street	40 50	73 57	+0 50	−0.5	0.9	8 52	3.9	4.8
Fort Washington Point	40 51	73 57	+0 55	−0.6	0.9	8 59	3.8	4.6
Spuyten Duyvil, W. of R. R. Bridge	40 53	73 55	+1 10	−0.8	0.8	9 13	3.6	4.4
Yonkers	40 56	73 54	+1 20	−0.8	0.8	9 25	3.6	4.4
Dobbs Ferry	41 01	73 53	+1 35	−1.0	0.8	9 40	3.4	4.1
Tarrytown	41 05	73 52	+1 50	−1.2	0.7	9 53	3.2	3.9
Ossining	41 10	73 52	+2 05	−1.4	0.7	10 08	3.0	3.7
Haverstraw	41 12	73 58	+2 15	−1.5	0.7	10 20	2.9	3.5
Peekskill	41 17	73 56	+2 45	−1.7	0.6	10 48	2.7	3.3
West Point	41 24	73 57	+3 25	−1.8	0.6	11 30	2.6	3.2
Newburgh	41 30	74 00	+4 00	−1.7	0.6	12 03	2.7	3.3
New Hamburg	41 35	73 57	+4 20	−1.6	0.6	0 01	2.8	3.4
Poughkeepsie	41 42	73 56	+4 45	−1.4	0.7	0 26	3.0	3.7

Fig. 13.—Reproduction of Part of Table 2 of Tide Tables, Showing How Secondary Ports Are Referenced to Data Shown Above.

Instruments for the Observation of Tides

As a means toward labor-saving and time-saving, automatic or self-registering devices give continuous observations of natural phenomena over long periods for use in planning engineering works. Thus, as the importance of series of tidal observations became apparent, the plain tide staff was replaced by automatic gauges.

The earliest automatic tide-gauge* was devised by an English civil engineer, Henry R. Palmer, to obtain a continuous record of fluctuations in the River Thames for getting the effect on the tidal regimen of the river by the removal of London Bridge, "free", as expressed by Mr. Palmer, "from the inaccuracies and doubts which the frequent and long-continued observations of individuals through nights and days must be liable."

* A paper descriptive of this gauge appears in *Philosophical Transactions*, Royal Soc., 1831.

The fundamental principles embodied in this first automatic gauge are still used and although there are many forms of these gauges, the principle is a simple one.* A float is connected by a fine bronze wire or tape with a self-recording pencil or pen, which moves at a reduced scale backward and forward across a paper record which is wound on a cylinder, either vertical or horizontal. This paper revolves by means of a clock movement, thus causing the pencil to trace a curve that accurately represents the tide for that locality.

The U. S. Coast and Geodetic Survey at its primary tide stations makes use of the automatic tide gauge shown in Fig. 14,† Its principal features are the motor clock which drives the main cylinder regulating the motion of the paper—a roll, 13 in. wide and 66 ft. long, sufficient for a month's record; the time-clock, similar to an ordinary striking clock, trips the recording pencil, making a mark on the tide graph or curve every hour; a system of pulleys and wires connects the mechanism with a tide-float which rises and falls in a well having a small opening for the entrance of the water. Usually the well is made of 2 by 12-in. planks, but at outside ocean stations it is composed of ordinary 12-in. water pipe. The opening in the well is about 1 in. to 1¼ in. in diameter for a float-well, 10 in. square, or 12 in. in diameter. The wave action is thus damped, but the hole is large enough to obviate any lag in the rise and fall as compared with the water outside the well.

To fill the need for a more portable gauge for field use, the Survey has recently developed a new automatic tide-gauge (Fig. 15). This can be used to advantage by hydrographic and tidal parties in the field at stations where it is necessary to obtain tidal observations extending over only a few days, or weeks, for the reduction of soundings taken during the survey, or for comparative purposes. In designing this instrument, the main objects were ease of installation and minimum size, commensurate with the desired accuracy.

The essential features of the gauge are its small size, its single drum on which the paper record is fastened by a spring clip, a single clock movement installed within the drum, a cast base fitting on top of a float-tube of stock iron pipe. This pipe, in addition to serving as a float-well, acts as a support for the instrument, thus obviating the necessity of providing an elaborate platform and a cumbersome float-well. This feature alone renders the gauge adaptable for use by field parties, particularly in remote localities where wharves and docks are not available. Another departure from the usual design of automatic gauge is the use of a spring instead of weights as a counterpoise for taking up the slack of the float wire on a rising tide.

The instrument is 10 in. square on the base and, with its weather-proof metal cover in place, is 11 in. high. This cover is held by a padlock, so that the mechanism cannot be tampered with when the gauge is installed in exposed places.‡

* Special types of pressure gauges have been designed for special purposes, such as the Fave "Maregraphe Plongeur," described in *Annales Hydrographiques*, 1908–10, pp. 383–437, and 1921, pp. 193–237. Various other types of automatic gauges have been described by R. A. Harris, in his "Manual of Tides," Pt. II, p. 480.

† This gauge is described in detail in *Special Publication No. 26*, "General Instructions for the Field Work of the U. S. Coast and Geodetic Survey," p. 113.

‡ This gauge is described in detail in *Special Publication No. 113*, U. S. Coast and Geodetic Survey, "A Portable Automatic Tide Gauge."

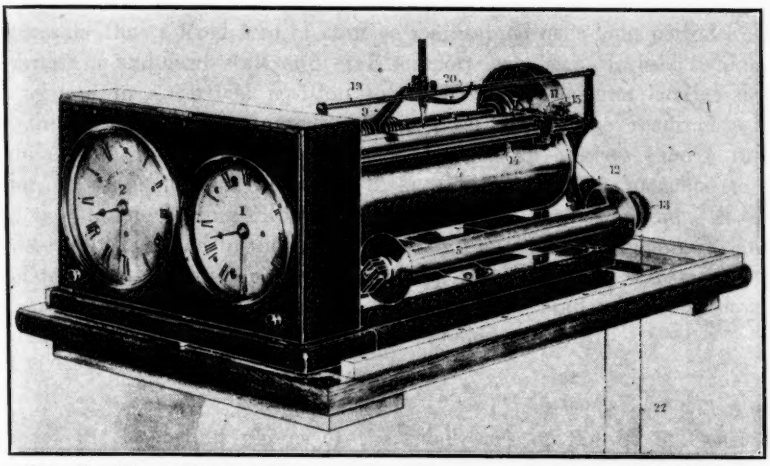


FIG. 14.—FRONT VIEW OF STANDARD U. S. COAST AND GEODETIC SURVEY MODEL AUTOMATIC TIDE GAUGE FOR USE AT PRIMARY TIDE STATIONS.

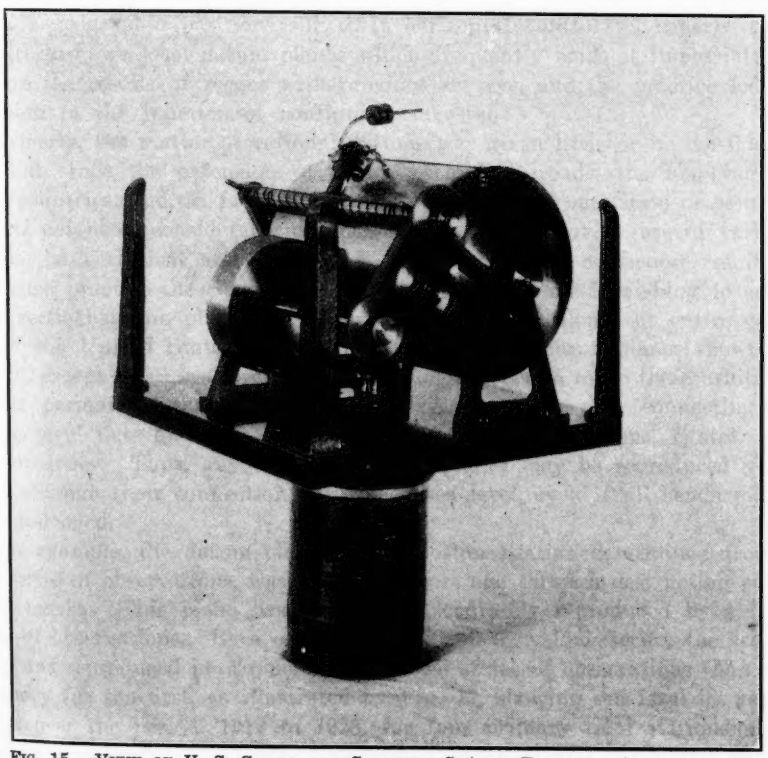


FIG. 15.—VIEW OF U. S. COAST AND GEODETIC SURVEY PORTABLE AUTOMATIC TIDE GAUGE (WITHOUT COVER).



For comparison of the marigraph curve with the height of the actual tide it is necessary that a fixed zero of staff be maintained over long periods. For this purpose a standard staff and staff support has been devised, which is firmly secured in a vertical position to a wharf or pile, and carries metal guides into which the graduated staff is inserted for the readings. For accurate observations the wave action is damped by a glass tube ½ in. in diameter, in which floats a thin cork disk. This glass tube, attached to the face of the staff by spring clips, is partly closed at the lower end. A brass stop is securely fastened to the back of the staff at an exact foot mark, and a brass plate is set into the top of the staff support, the stop on the staff resting firmly on this plate as the observation is made. The staff support is covered with sheet copper in waters infested with teredos.

Mean Sea Level*

One of the important phases of the tidal work of the U. S. Coast and Geodetic Survey is the determination and establishment of datum planes based on tidal definition. Surveyors and engineers find necessity for these planes as rational datums for vertical control of their operations just as they have need for geographic positions for their horizontal control. Formerly they used arbitrary or local datum planes which frequently made it impossible to compare the results of recent with previous surveys, and the practice led to confusion in the junction of contiguous surveys.

Formerly, the matter of rational datums was given little or no consideration, but since the extension of highways and railroads, the mapping of whole countries, and, in fact, the joining of surveys of one State or country with its neighbor, considerable confusion is brought about by use of various datums, both vertical and horizontal. To obviate the confusion resulting from such practice the Survey has been establishing and furnishing to engineers vertical datum planes, based on tidal definition, along the entire coast line of the United States. As an example, all the datum planes shown in Fig. 16, except mean sea level are arbitrary. In order to make these arbitrary datums permanent throughout the country, the Survey is connecting by precise level nets all those used in the past by organizations, States, and municipalities. Thus, any of these various planes may be reproduced years hence through their connection with mean sea level, even if all bench marks were destroyed.

For example, the datum plane at the Boston Station determined from a long series of observations, was lost some years ago through destruction of all bench marks. This plane has since been accurately reproduced by a later series of observations. By a comparison with near-by long series, the second plane was reproduced by a much shorter local series of observations than was necessary for the first, as illustrated by Fig. 17, showing sea level by yearly means over the period, 1912 to 1923, for four primary tidal stations along the Atlantic Coast.

* For a more detailed discussion, see "Mean Sea Level and Its Variations," by H. A. Marmer, *Annals*, Assoc. of American Geographers, September, 1925.

This graph brings out the interesting fact that the causes for a yearly variation from a mean sea level are not local; but extend, for example, along the entire Atlantic seaboard. For 1919 (the year of largest variation), the

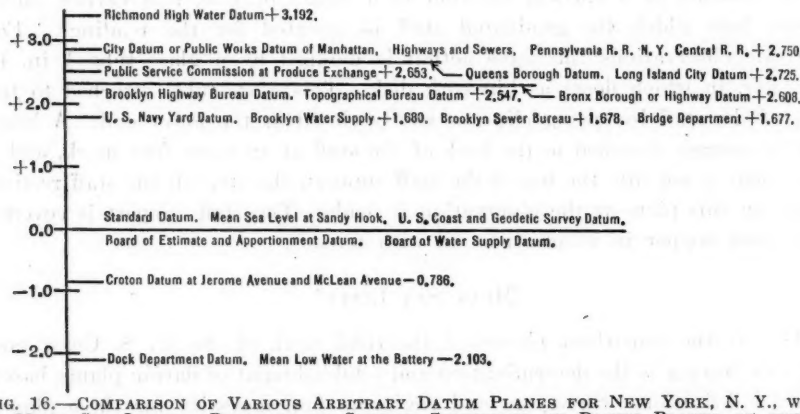


FIG. 16.—COMPARISON OF VARIOUS ARBITRARY DATUM PLANES FOR NEW YORK, N. Y., WITH MEAN SEA LEVEL. PREPARED BY SPECIAL COMMITTEE ON DATUM PLANES OF THE MUNICIPAL ENGINEERS OF THE CITY OF NEW YORK (SEE ITS *Proceedings*, 1915).

sea-level determination at Portland was 0.13 ft. above the long-period mean sea level; 0.22 ft. above at Fort Hamilton; 0.17 ft. above at Atlantic City, N. J.; and so on down the entire coast to Fernandina, Fla.

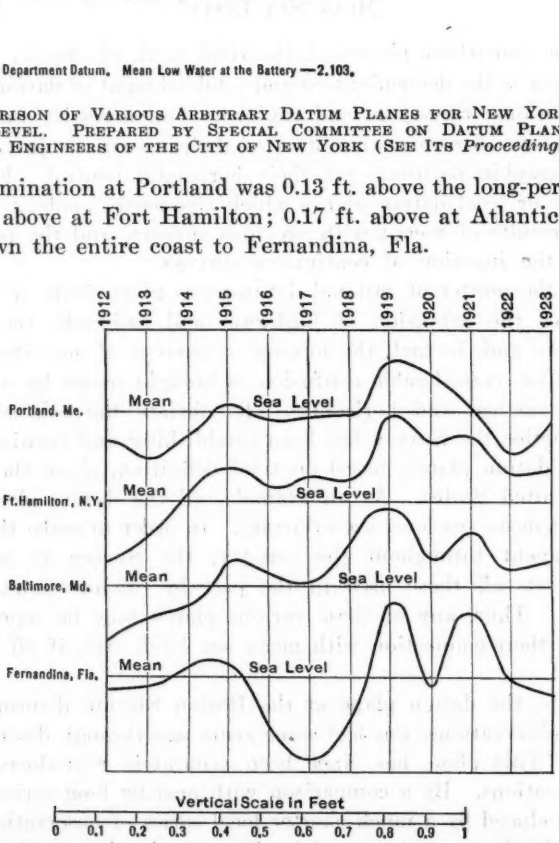


FIG. 17.—GRAPH OF SEA LEVEL DETERMINED FROM YEARLY MEANS (1912–23) AT PRIMARY TIDE STATIONS, COMPARED WITH LONG PERIOD MEAN SEA LEVEL.

The uniformity of these variations over a large area is used for practical purposes in the accurate determination of mean sea level from a short series by comparison with long series at near-by stations. For example, the nearest

value to a mean sea level at Boston may be desired from local tidal observations during 1919.

Since it is clearly evident from Fig. 17 that sea level along the entire Atlantic Coast of the United States was above the long-period mean sea level during that year, this may safely be assumed to have been true also at Boston. The mean of the sea levels that year at Portland and at Fort Hamilton was 0.17 ft. above the long-period mean. Including Atlantic City, the average of the three also gives the value, 0.17 ft., above. Taking the average of the variations from the mean for several near-by ports, it is quite probable, therefore, that for all practical purposes a very close value of mean sea level for Boston may be obtained by subtracting 0.17 ft. from the determination from the observations made locally in 1919.

As a test of the accuracy of a datum obtained by reference with primary stations, the comparison may be made separately with two near-by stations at which long series have been observed.* For use in checking a line of precise levels a mean sea level determination was desired for Anacortes, Wash. A tide-gauge was established and maintained for a three-year period; an uncorrected mean sea level determination of 19.73 ft. on the tide staff was obtained from these observations. The Canadian Government furnished the U. S. Coast and Geodetic Survey the hourly readings for Victoria, B. C., for the same three-year period. These data from Victoria, with similar data from the United States Tide Station at Seattle, were used for correcting the three-year determination at Anacortes to the mean sea level datum determined from long series at the other two stations by comparison of simultaneous observations.

Victoria is distant about 70, and Seattle, about 35, nautical miles from Anacortes. The following are the results of correcting the Anacortes datum by reference to the two ports by separate computations:

Anacortes mean sea level on tide staff determined by averaging the hourly readings of the tide from 3 years of observations	19.73 ft.
Anacortes mean sea level on tide staff, corrected to a 17-year determination of mean sea level at Victoria, B. C., from comparison of 3 years of simultaneous observations.....	19.81 ft.
Anacortes mean sea level on tide staff, corrected to 23-year determination of mean sea level at Seattle, Wash., from comparison of 3 years of simultaneous observations.....	19.82 ft.

A shorter period than a year may be used with fairly satisfactory results. It was desired to make temporary use of the determination of a mean sea level datum for Anacortes before the full three years of observations had been obtained, so the mean sea level as determined at Anacortes for 19 months' observations was corrected to the 22-year determination at Seattle with the following results:

Anacortes mean sea level on the tide staff, corrected to a 22-year determination of mean sea level at Seattle from comparison of 19 months of simultaneous observations....	19.84 ft.

From these examples it is evident that primary tidal stations with long series, along a coast line are of prime importance to tidal control of regions of which they are representative.

* *American Journal of Science*, April, 1926.

Therefore, primary determinations of mean sea level at various places along the coasts are important for comparison with short series of tide observations at near-by stations to reduce the results of the short series to mean values. These primary observations should continue over long periods, to bring out all the variations in mean sea level and to obtain mean values for all tidal constants. A period of 19 years is considered a full tidal cycle and results over such an interval are regarded as constituting mean values. Therefore, sea level derived from 19 years of observations has been generally taken as constituting a primary determination of mean sea level. Unfortunately, observations have not been continued long enough at any station to determine the difference in sea level as found from two or more full 19-year periods.

Thirty years of observations are available for the primary tide station at Fort Hamilton, N. Y. For the two overlapping 19-year periods—1893 to 1911 and 1904 to 1922—a value for sea level of 5.96 ft. on the staff is derived for the first period and 6.02 ft. for the second period.* The mean value for the entire period of 30 years is 6.00 ft. on the tide staff.

For practical purposes, however, it is hardly possible to obtain 19 years of observations at a station. Moreover, it has been found that the principal variations in sea level from year to year are cyclic in character, the level rising for a period of several years and falling for a period of years. Observations are not available for a sufficient number of years, however, to determine accurately the periods of the cycles, but there appears to be one with a period of 4 to 5 years and also one with a period of 8 to 9 years.† The variation with the period of about 9 years is quite prominent and for practical purposes, therefore, sea level determined from 9 years of tide observations may be considered as a primary determination of mean sea level.

Through the work of the Geodetic Division of the U. S. Coast and Geodetic Survey a network of precise level lines has been carried to many parts of the interior of the United States (Fig. 18). These lines have as their initial and "tie-in" points a mean sea level plane determined at the primary stations along the seaboard. Eventually every locality will be within 50 miles of a bench mark on this net. On the Atlantic Coast long series of observations are being obtained at Portland, Boston, New York, Philadelphia, Atlantic City, Baltimore, Charleston, S. C., and Daytona, Fla.; on the Gulf Coast, at Key West and Pensacola, Fla., and Galveston, Tex.; and on the Pacific Coast, at La Jolla, Calif., San Francisco, Seattle, Ketchikan, Seward, and Valdez, Alaska. All the observations at these stations are connected with permanent bench marks for the use of engineers locally, or for connection with precise level nets.‡

* U. S. Coast and Geodetic Survey *Special Publication No. 111*, "Tides and Currents in New York Harbor," p. 32.

† U. S. Coast and Geodetic Survey *Special Publication*, "Tidal Datum Planes," (in press), by H. A. Marmer. (See this publication for full discussion of determination of mean sea level and other tidal datum planes.)

‡ In this connection the Coast and Geodetic Survey has recently issued a publication containing the descriptions and elevations of about 1 800 tidal bench marks in New York State, and also one of the District of Columbia containing about 85 descriptions and elevations of tidal bench marks in the City of Washington. These publications will be followed by similar ones for other sections of the coasts and should prove of considerable value to engineers engaged on the construction of docks, bridges, harbor and river improvements, and other marine work.

Datum Planes

The simplicity of its definition, as well as the certainty with which it may be reproduced at some future time, gives value to the tidal datum as a plane of reference. In general, the most satisfactory plane is that of mean sea level, since the other tidal planes may be derived from it by knowing the range of the tide and its diurnal inequality.

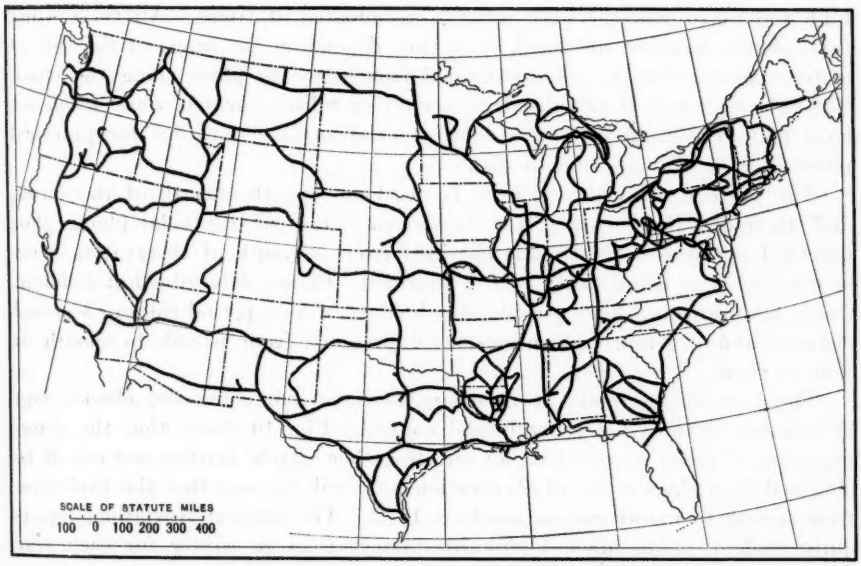


Fig. 18.—Precise Level Net of the United States. This Level Net Is Based on Mean Sea Level Datums Established at Primary Tide Stations.

Mean sea level may be defined as that level about which the tides oscillate. Since no fixed plane exists in Nature, it is necessary to devise some means for determining and for giving concrete value to this plane, above and below which the tide oscillates. The only means for making this determination is by observations of the rise and fall of the tide. It is impracticable to make use of the entire tidal curve, so that, instead of integrating the full curve, recourse is had to averaging the hourly heights of the tide. These hourly heights, tabulated from the record of the observations, are averaged over the entire period. The result is known as mean sea level, and the final accuracy of its determination is defined by the length of the series of observations. The series must be of sufficient length to have passed through varied meteorological conditions and atmospheric pressures, or must be corrected by comparison with other stations having periods of observation of sufficient length to have passed through these varied conditions.

For navigation a low-water plane is the most satisfactory because by its use the soundings on a chart will show the least depth to be expected, regardless of the state of the tide at the time the soundings were made in the hydrographic survey. A plane of mean low water, therefore, has been adopted for

the whole of the Atlantic Coast; on the Pacific Coast, because of the diurnal inequality in the tides, the plane of mean lower low water is used; that is, all soundings shown on charts of the U. S. Coast and Geodetic Survey have been reduced, no matter what the stage of the tide at the time the survey was made, to the plane of mean low water on the Atlantic Coast, and, in general, to mean lower low water on the Pacific Coast and Alaska.

For other engineering purposes planes determined by high waters, higher high waters, or extreme high waters, are adopted at times. All these other tidal planes, however, are local, since they depend on the range of the tide at a given place; that is, when extended from place to place along the coast, these planes would, if visualized, be a wavy or rolling surface, while mean sea level may be considered as an equi-potential surface, forming for practical purposes a continuous smooth plane.

The purpose for which it is to be used governs the length of the series, and, therefore, the accuracy, for the determination of any tidal plane. For practical purposes a tidal plane obtained from a month of observations may be considered as being fairly well determined. Planes determined at different times, however, may differ considerably because of long period tides or seasonal changes and differences of atmospheric pressure from month to month or year to year.

These conditions make it necessary to have long-continued observations at primary stations for general tidal control. Fig. 19 shows that the determination of mean sea level is not an altogether simple matter, nor can it be obtained by a short series of observations. It will be seen that the variations from month to month are as much as 1½ ft. The seasonal variation, too, is quite evident, being much higher in summer than in winter for each year throughout this period.

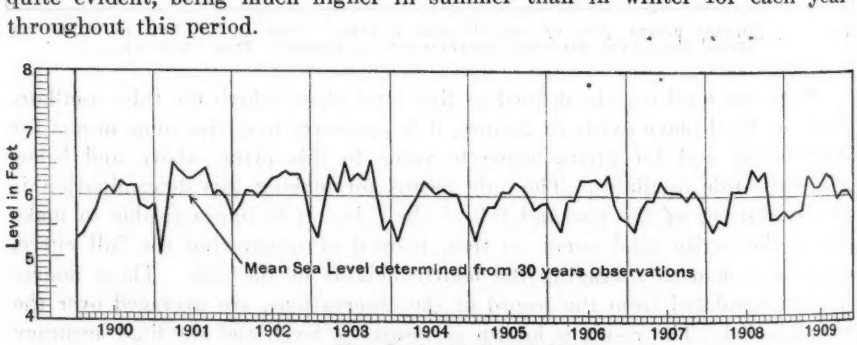


FIG. 19.—DIAGRAMMATIC REPRESENTATION OF THE PLANE OF MEAN SEA LEVEL PLOTTED FROM MONTHLY MEANS, 1900 TO 1909, AT FORT HAMILTON, N. Y.

Sea level as determined by a series of observations extending over only a month may differ as much as 1 ft. or more from that determined for another month. Even the level as determined by observations extending over a year may differ as much as ¼ ft. from a level determined during another year, so that for geodetic and other scientific work, even a year is not sufficient.

There is a distinction between the terms, "mean sea level", "mean tide level", and "mean river level". Mean sea level is obtained by integrating the

tide curve, at least to the extent of taking the average height of the curve at frequent intervals; mean tide level is the plane exactly half way between the high and low waters and is obtained by taking the half sum of all high and low waters; and mean river level is the mean height of the river surface as influenced by the tide, freshets, and bottom slope, and obtained in the same manner as mean sea level.

If the tide curve were a simple sine curve the plane of mean sea level and mean tide level would coincide; actually the larger part of the curve may be above or below the mean level of the water. The difference between the two, however, is small, and if the series of observations is short the two planes may be considered for practical purposes as coincident. On the Atlantic Coast the plane of mean sea level lies above the plane of mean tide level—at Portland, 0.03 ft.; at New York, 0.05 ft.; at Atlantic City, 0.03 ft.; and at Fernandina, 0.10 ft. On the Pacific Coast, mean sea level is the lower of the two—at Seattle, 0.01 ft.; at San Francisco, 0.06.; and at San Diego, 0.05 ft. below. The planes of mean sea level and mean river level coincide at the mouth of a river, but gradually separate toward the head.

A very accurate determination of this natural standard, mean sea level, is also of considerable value to the scientist in furnishing the only means for a quantitative measure of a possible emergence or subsidence of land areas. When the determination of mean sea level has been extended over centuries quantitative evidence will be available as to the stability of coast lines of continents by reference to permanent bench marks connected with this mean sea level datum. The primary tide stations of the U. S. Coast and Geodetic Survey are placed so as to be representative of certain regions and to furnish tidal control for these regions for practical purposes. In addition to serving these purposes, however, the continuous observations covering a long period of years bring out the variations in mean sea level, thus furnishing data for a quantitative determination of any relative change in elevation of land and water.

Earthquake Waves

The "tidal wave" of the press is a misnomer. While these waves are superimposed at times on curves of tide record, vitiating the records during the time they are recorded, they are in no sense due to tidal forces but rather to seismic disturbances.

If the submarine disturbance be due to a horizontal slip along a fault line, practically no distortion occurs in the regular tide curve; but an earthquake caused by vertical displacement produces waves of considerable magnitude. The San Francisco earthquake of 1906 was caused by a horizontal slip along an old fault line, and the tide record at Fort Point in the Golden Gate showed very little disturbance. In Japan after the disastrous earthquake of September, 1923, a resurvey of Sagami Bay indicated that a considerable vertical displacement had taken place and a wave more than 40 ft. high was reported in a part of that Bay.

Waves 80 ft. in height have been reported in some places near the origin of seismic disturbances. These waves are propagated for considerable dis-

tances over the oceans, at times reaching the shores of other continents, although with constantly diminishing amplitudes. Very destructive seismic sea waves have occurred in regions subject to earthquake disturbances throughout historical times; among others, the destruction of Helike, Greece, in 373 B. C.; at Callao, Peru, in 1724 and 1746; at Lisbon, Portugal, in 1755; Arica, Chile, in 1868; Iquique, Chile, in 1877; and on the Japanese Coast in 1896 and again in 1923.

The sea waves caused by the earthquake which occurred on November 11, 1922, off the coast of Chile were recorded on the tide records at the Survey's primary tide stations at Honolulu, San Diego, San Francisco, and at Zamboanga, Philippine Islands. The marigrams from these stations are shown in Fig. 20, all reduced to Greenwich civil time, November 11-12, 1922. It will be noted that the curve at the beginning of the tidal graph for each station has only minor fluctuations caused by wind wave effects; the remainder of the graphs, however, have distinct fluctuations of considerable amplitude.

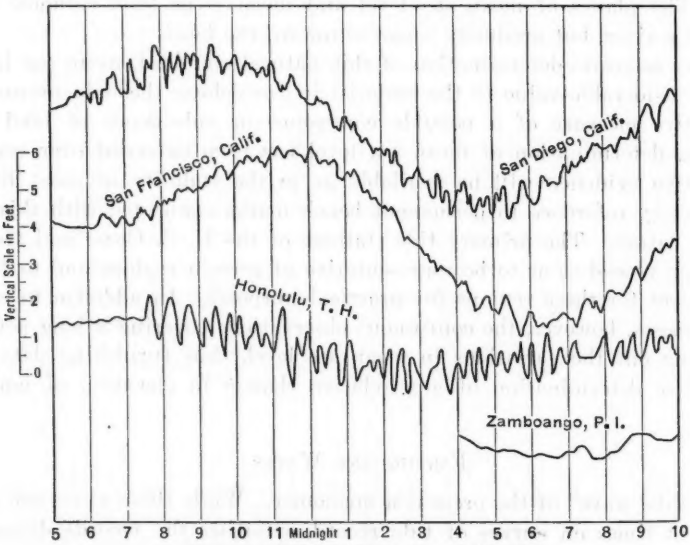


FIG. 20.—RECORDS ON THE MARIGRAMS OF TIDE STATIONS AT SAN DIEGO, SAN FRANCISCO, HONOLULU AND ZAMBOANGA, OF THE SEISMIC SEA WAVES FROM THE EARTHQUAKE WHICH OCCURRED OFF THE COAST OF CHILE, NOVEMBER 11, 1922.

The records from the seismograph stations at Honolulu, Sitka, Tucson, Cheltenham, and Vieques, indicate that the earthquake occurred on November 11 at 4:33 A. M. (Greenwich civil time), and that it originated off the coast of South America in the vicinity of Carrizal, Chile, at about 28° South Latitude. The first fluctuation on the record at San Diego (Fig. 20), began on November 11, at 5:30 P. M. (Greenwich civil time). A maximum amplitude was reached about 2 hours and 20 min. later, with a range of 1.2 ft. and with a period of from 30 to 40 min. Assuming that the propagation of the wave began at the origin of the earthquake simultaneously with the first earth

tremor, its time of transmission from Carrizal to San Diego (4 480 miles) is 12 hours and 57 min., or at the rate of 346 miles per hour for an average derived depth of water of 1 750 fathoms. Table 1 gives similar comparative data for four stations.

TABLE 1.—Sea Wave from Earthquake of November 11, 1922, off the Coast of Chile.

Station.	Distance from Carrizal, in nautical miles.	Arrival of wave, (Greenwich civil time).	Time of Transmission of Wave.		Nautical miles per hour.	Derived depth of ocean,* in fathoms.
			Hours.	Minutes.		
Honolulu, Hawaii.....	5 850	Nov. 11, 7:30 P. M.	14	57	390	2 250
San Diego, Calif.....	4 480	Nov. 11, 5:30 P. M.	12	57	346	1 750
San Francisco, Calif..	4 890	Nov. 11, 6:20 P. M.	13	47	356	1 880
Zamboanga, Philippine Islands.....	9 350	Nov. 12, 6:45 A. M.	26	12	357	1 890

* Mean depths computed from Table 51, "Manual of Tides," by R. A. Harris, Pt. III.

Tidal Power

Except in rare cases, the engineer is not directly interested in the matter of tidal power, that is, the energy in the gravitational force of moon and sun exerted on the rotating earth, expressed in concrete form in the ceaseless rising and falling of the water of the oceans. While a study of this power is not a function of the U. S. Coast and Geodetic Survey, its Division of Tides and Currents has kept in touch with the subject in order to answer the requests for information addressed to it as the tidal authority in America.

The economical harnessing on a large scale of the energy in the tides has been the dream of the engineer for centuries; but, unlike fire, steam, the water power of streams, and electricity, it is the only force of Nature not enslaved by man. As compared with coal and oil, the initial cost of the necessary construction and installation has rendered the use of tidal power generally out of the question. When nations find it essential to begin by some means to conserve the fuel supply, engineers may find inexhaustible resources in the tides if they are successful in economically harnessing this power. The future may then see projects, introduced by countries or States, situated favorably from a standpoint of wide range of tide. By that time, however, the engineering difficulties will have been completely overcome and the necessary machinery developed to a high state of efficiency.

Plans for the utilization of power from the tides have been rejected by many engineers as involving too great initial cost in the construction of reservoirs in proportion to the power obtained. It must be borne in mind, too, that in practice it is not possible to obtain the full theoretical power, nor to utilize the vast power of all the oceans, but only that small fraction of tidal power available near the coast where natural facilities permit empounding small parts of the sea and constructing power plants.

The Civil Engineering Department of the British Ministry of Transport has prepared plans* for the utilization of tidal power in the estuary of the River Severn, where the range of tide is from 20 to 30 ft. The English propose building a dam for empounding the waters of the Severn, using it also as a bridge for railway and vehicular traffic, connecting South Wales with the remainder of the country. The combining of the two projects will partly lower, of course, the initial cost chargeable to the tidal project; and this initial cost has previously been considered the vital objection.

The utilization of tidal power has also received attention from French engineers, who have proposed a number of plans. The northwest coast of France lends itself to such schemes. The mean ranges along the coast bordering the Bay of Biscay vary from 12.7 ft. at the mouth of the Gironde River to 14.3. ft. at Ushant Island; and along the coast on the English Channel, from 16.6 ft. at Ile de Bas to 26.8 ft. at St. Malo, decreasing to the northward to 16.2 ft. at Cape Griznez, Strait of Dover. A more recent project has been proposed in America in the plan to utilize the large range of tide in Passamaquoddy Bay, New Brunswick, and Cobscook Bay, Maine.

Heretofore, the harnessing of the tides for power purposes has been approached on a much smaller scale than these modern proposals. Tide mills of small power for grinding corn have been in existence in Spain, on the Thames in the Isle of Wight, and on the Bay of Fundy, but having an efficiency of only about 10 per cent. In all these cases the range of the tide is considerable, a necessary factor for the success of any tidal power plan. Means for furnishing power during the time the tide is rising or at a stand is another important factor in tidal power development. In addition, the variations in range during a lunar month add complexities to the problem.

Should engineers be successful in economically harnessing the tides, their use on the Atlantic Coast of the United States would be limited to Northern New England, because of comparatively small ranges of tide elsewhere. At Calais, St. Croix River, Maine, the mean range is 20.0 ft., decreasing gradually southward along the coast to 10.5 ft. at Bar Harbor, Mount Desert Island. In Penobscot Bay, the mean range varies from 9.1 ft. at Head Harbor, Isle au Haut, to 13.1 ft. at Bangor, Penobscot River. South of Penobscot Bay the mean range is less than 10 ft. along the entire Atlantic Coast, with the exception of Wellfleet, Cape Cod, where it is 10.7 ft.

Estuary Tides

Practically all research work on estuary tides has been done by the tidal mathematicians for the purpose of increasing the accuracy of predictions of tides for tide tables. A fertile field remains open for the engineer in a study of the changes in the tidal regimen in an estuary or in a river resulting from changes in topography and depth in harbor and river improvements, in order that dependable quantitative predictions may be made for proposed projects.†

* "One Million Horsepower from Tidal Energy," *Scientific American*, January 22, 1921.

† L. Bonnet's "Contribution a l'Etude Théorique des Fleuves a Marée et Application aux Rivières a Marée du Bassin de l'Escaut Maritime," goes into the theories of estuary tides and their application to the River Scheldt and tributaries.

Apparently the factors entering into the problem differ so widely for each body of water that each case requires individual treatment.

In New York Harbor, for example, where long series of observations are available covering the period during which extensive dredging operations have been carried on, the indications are that little change has occurred in the tidal regimen. The mean ranges, as derived from two overlapping 19-year series, from 1893 to 1922, in 10-year groups,* indicate an increase of 0.1 ft. in a decade, due to the improvement to Ambrose Channel between 1901 and 1914. This increase in range can be ascribed to the widening and deepening of this channel, thus allowing freer access of the tidal wave to New York Harbor.

On the other hand the dredging and development of Delaware Bay and River have made a considerable change in the tidal régime, but of a different effect; instead of an increase the tides of the Delaware have been decreased in range 0.7 ft. in some places,† and the time of the tide retarded by more than ¼ hour. This decrease has been accompanied by the raising of the mean low-water plane, the high-water plane remaining practically constant.

In Fig. 21 are shown the tidal ranges in the entire Delaware Waterway from observations prior to 1890, and from 1900 to 1924. It will be noted that no change occurred in the range from the mouth of Delaware Bay to Woodland Beach. In Delaware River, however, where extensive improvements were made during the period, 1890 to 1900, the curves diverge as much as 0.7 ft. Just below Trenton, N. J., the curves intersect again, and the range at Trenton is now 1 ft. greater than formerly.

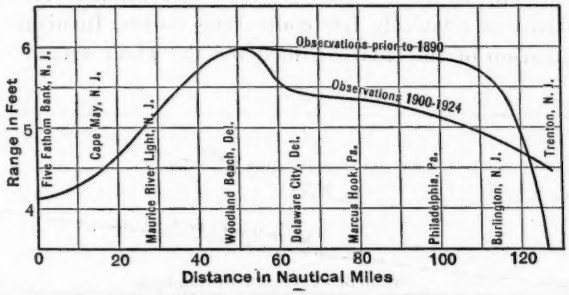


FIG. 21.—RANGE OF TIDE, DELAWARE BAY AND RIVER FOR PERIOD PRIOR TO 1890, COMPARED WITH RANGES FOR PERIOD 1900 TO 1924, AFTER EXTENSIVE HARBOR IMPROVEMENTS.

The physical reasons for this change may be ascribed to the fact that the dredging was not confined to a deepening of the channel, but involved a considerable widening of the waterway by the removal of shoal areas, allowing for a much greater cross-section to accommodate the original tidal prism entering the lower bay where no extensive developments have been made, and also tending to decrease friction in the channel.

* See U. S. Coast and Geodetic Survey *Special Publication No. 111*, "Tides and Currents in New York Harbor," pp. 45–46.

† See U. S. Coast and Geodetic Survey *Special Publication No. 123*, "Tides and Currents in Delaware Bay and Tributaries." (In Press.)

The increase of the range at Trenton may be ascribed to the same causes that contributed to the increase of the tidal range in New York Harbor. The deepening and widening of the lower channels has given freer access of the tide to the upper reaches of the river and, therefore, increased the range. This accommodation of the upper reaches of the river to a greater part of the total tidal prism entering the whole waterway is another factor, too, in the decrease of the range in the intermediate part of the waterway.

So far all interpretations of data from current and tide surveys have been necessarily of a qualitative nature since sufficient data are not yet available for quantitative studies. It may be concluded, then, that in general the deepening and widening of the channels leading from the ocean into an estuary will result in a slight increase in the tidal range of the estuary; that similar operations in upper channels with no increase of the entrance cross-section will result in a decided decrease in range in the improved area, but a decided increase in range in the upper reaches of the river.

In the practical study of the tidal regimen of an estuary or river not broken by abrupt falls, and in the interpretation of the tidal data from scattered observations, a graph similar to Fig. 22 is of considerable value. This diagram has been prepared for purposes of illustration and assumes the slope of the Hudson River to be uniform (which undoubtedly is not strictly correct). Spirit-level connections between the line of precise levels that has been run from New York to Troy and various tide observations made over long periods, would undoubtedly indicate that the river profile is not strictly a straight line. Because of freshets evidently the accurate variations from a straight line can be brought out only by long series of tide observations. However, a graph from observations at relatively few controlling points, furnishes a sufficiently exact approximation of the tidal regimen for the whole river.

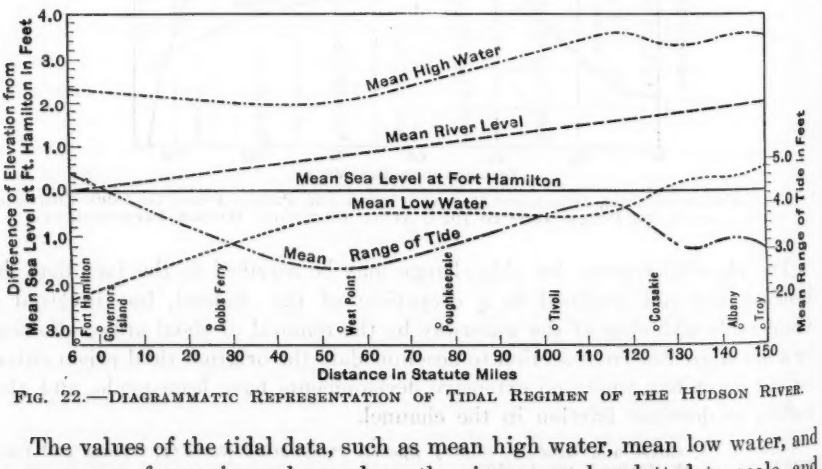


FIG. 22.—DIAGRAMMATIC REPRESENTATION OF TIDAL REGIMEN OF THE HUDSON RIVER.

The values of the tidal data, such as mean high water, mean low water, and mean ranges for various places along the river, may be plotted to scale, and smooth curves drawn through the points. In this manner a fair approximation of the tidal regimen is obtained for the whole length of the river from observa-

tions at relatively few controlling points. For example, from Fig. 22 the mean river level at West Point is 0.75 ft. above the plane of mean sea level; 1.01 ft. at Poughkeepsie; 2.06 ft. at Troy, etc.

The velocity, V, of a progressive tidal wave, such as occurs in the Hudson River and Chesapeake Bay, will be:

$$V = \sqrt{g\,h} = 5.67\ \sqrt{h}\ \text{ft. per sec.}$$

in which, $g = 32.17$ ft. per sec. (acceleration due to gravity), and $h =$ depth of water for the average cross-section, in feet.

In order to convert feet per second into nautical miles per hour, multiply by 0.592, and,

$$V = 3.36\ \sqrt{h}\ \text{nautical miles per hour}$$

The time required for the wave is,

$$t = \frac{17.87}{\sqrt{h}}\ \text{min. per nautical mile}$$

$$t = \frac{15.51}{\sqrt{h}}\ \text{min. per statute mile}$$

The character of the wave in a body of water, whether progressive or stationary, can be determined by computing the rate of advance of the tide by this formula and comparing this computed rate with tide observations along the waterway. If the actual rate is considerably in excess of the computed rate and if, in addition, the range as shown from the observations increases toward the head, it may be concluded that the body has a stationary wave tide.

The character of the tide wave may also be determined from the occurrence of the times of the slack and strength of the currents. In a body having a progressive tidal wave the time of slack water occurs about the time the tide is near mean sea level, and the strength of the current occurs near the time of high or low water; whereas, with a stationary wave, slack-water occurs near the time of high or low water, and the strength of the current. at about the time the height of the tide is near mean sea level.

The tide wave in the Hudson is of the progressive type and the range decreases toward the head. In the vicinity of West Point a sharp decrease in range occurs, due to the marked increase in the depth of waterway and the absence of any obstructions to the free passage of the tidal wave. After its passage through this deep, open stretch the tidal wave is obstructed by shoaler water and lateral shoals, and the range, therefore, increases somewhat, with a more marked increase in the stretch between Tivoli and Coxsackie where numerous and extensive middle-grand shoals still further impede the progress of the tidal wave.

The general theory that the range decreases progressively toward the head in an estuary having the progressive type tide and increases in one having the stationary wave type presupposes a body of uniform width and depth. The reason for any departure from this theory will generally be found in the physical characteristics of the body, as seen in the case of the Hudson.

As another example, the tide in Chesapeake Bay is of the progressive type. From the tide tables the mean ranges of the tide at points directly opposite on the western and eastern shores from the Capes to the mouth of the Patapsco River, are given in Table 2.

TABLE 2.—MEAN RANGES OF TIDE IN CHESAPEAKE BAY.

Western Shore.	Range, in feet.	Eastern Shore,	Range, in feet.
Cape Henry	2.8	Cape Charles	3.0
Near Point Comfort	2.2	Cherry Stone Light	2.3
Stingray Point	1.4	Mouth, Occohannock Creek	1.8
Smith Point Light	1.2	Tangier Light	1.6
Point Lookout	1.3	Holland Island Light	1.6
Cedar Point	1.2	Barren Island	1.5
Chesapeake Beach	0.9	Sharp's Island	1.3
Herring Bay	0.9	Poplar Island	1.2
Sandy Point	0.8	Love Point	1.1
Seven Foot Knoll	0.9	Tolchester Beach	1.3
Average mean range	1.36	Average mean range	1.67

The range decreases progressively up the Bay to Seven Foot Knoll on the western shore and Tolchester Beach on the eastern shore where there is a slight increase. This increase here is to be expected, since the tide wave is retarded just below these places by the constriction of the channel and by the obstruction to the progress of the wave afforded by the shoal areas off the mouth of the Patapsco River. The differences in the ranges on the two shores of the Bay will be explained hereafter.

Still another example is the Potomac River. As shown in Table 3, this river may be divided into sections as to increase or decrease in range toward the head.

TABLE 3.—CHANGES IN TIDAL RANGE IN ASCENDING POTOMAC RIVER

FIRST SECTION—INCREASE IN RANGE.	
Point Lookout	1.3 ft.
Piney Point	1.6 "
Ragged Point	1.8 "
Blackiston Island	1.9 "
SECOND SECTION—DECREASE IN RANGE.	
Colonial Beach	1.6 ft.
Lower Cedar Point	1.4 "
Mathias Point	1.2 "
Maryland Point	1.1 "
THIRD SECTION—INCREASE IN RANGE.	
Brents Point	1.2 ft.
Quantico Creek Entrance	1.4 "
Indian Head	1.7 "
Marshall Hall, Md	2.1 "
Alexandria, Va	2.8 "
Washington, D. C	2.9 "

A chart of the Potomac will show that the first section is just below Kettle Bottom Shoals, which furnishes a considerable obstacle to the advance of the wave. It is to be expected that the ranges would increase below this point.

The second section comprises a comparatively deep, free channel and the tidal ranges, therefore, decrease as might be expected.

In the third or upper section the river is practically filled with extensive shoal areas and the channel reduced to a mere thread (relatively). The ranges increase, therefore, toward the head due to the retardation of the tidal wave.

The Bay of Fundy, already mentioned, furnishes a good example of the increase in range toward the head in a body having a stationary wave tide. Table 4 shows the ranges directly opposite on both shores of the Bay from its mouth to its head (taking as the mouth a line between Belfast and Jebogue Point).

TABLE 4.—MEAN RANGES IN TIDE IN BAY OF FUNDY.

Western Shore.	Range, in feet.	Eastern Shore.	Range, in feet.
Belfast, Me.	9.7	Jebogue Point, N. S.	14.0
Eastport, N. B.	18.2	Annapolis, N. S.	25.1
St. Johns, N. B.	20.9	Port George, N. S.	27.8
Moncton, N. B.	41.2	Minas Basin, N. S.	44.2
Average mean range.	22.5	Average mean range.	27.8

The average mean range on the eastern (right) shore in the Bay of Fundy is greater by 5.3 ft. than on the western (left) shore, as found for Chesapeake Bay.

Fig. 23 shows the ranges on both shores of the Bay of Fundy. The upper curve is drawn directly through the plottings of the mean ranges at several ports on the Nova Scotia shore as taken from the tide tables; the lower curve is for the Maine-New Brunswick shore. These two curves of ranges differ at the mouth of the Bay by about 4 ft., increasing to about 6 ft. half-way up and decreasing to about 3 ft. at the head where a narrowing of the Bay takes place. The greater range is on the right shore.

Fig. 24 illustrates the same feature in Delaware Bay. Beginning at Five Fathom Bank Light Vessel, just outside the entrance to the Bay, the two curves representing the values of the mean ranges on the two shores separate, the greatest difference of 0.8 ft. occurring about half-way up the Bay. Toward the head the two curves again approach each other. Here, too, the greater range is on the New Jersey side, the right shore facing up the Bay.

This interesting phenomenon of the difference in range on the two shores of an estuary is due to the deflecting force of the earth's rotation, acting to the right (and so causing a greater range on the right shore) in the Northern Hemisphere and to the left in the Southern Hemisphere.* The theoretical

* For formula, see *Special Publication No. 111*, U. S. Coast and Geodetic Survey, "Tides and Currents in New York Harbor," by H. A. Marmer, p. 68, Govt. Printing Office, 1925.

value, in feet, by which the ranges on the two shores of a tidal stream differ, is represented approximately by the formula, $\frac{3\,v\,d\,\sin\,\phi}{g}$, in which, v is the velocity of the water, in knots; d, is the width of the body, in nautical miles; ϕ is the latitude; and g is the acceleration of gravity, in feet per second (approximately 32.2).

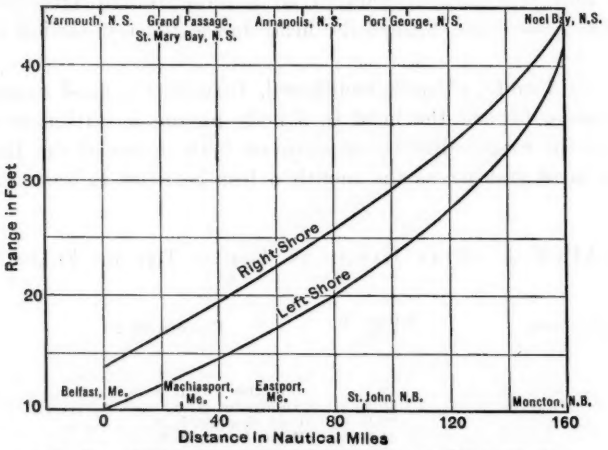


FIG. 23.—COMPARISON OF THE RANGES OF TIDE ON THE TWO SHORES OF BAY OF FUNDY.

Another character of tide occurs in short straits connecting two independently tided bodies of water. The movement of the water is largely hydraulic in character. The East River furnishes a good example of this type.* It will be taken up here briefly. Here the tide is derived from the tide in New York Bay, which is due to a progressive wave, and from that in Long Island Sound which is largely due to a stationary wave. These tides differ 3 hours in time and about 3 ft. in range.

FIG. 24.—COMPARISON OF RANGES OF TIDE ON TWO SHORES OF DELAWARE.

At times the peculiar condition resulting has been ascribed to the interference of two tide waves—one from New York Bay and one from Long

* The tide in the East River is covered in detail in U. S. Coast and Geodetic Survey *Special Publication No. 111.*

Island Sound—whereas it is merely a changing height of water brought about by the changing levels in the two bodies of water which it connects. The stage in the East River at any time is conditioned on the relative heights of the water in the Upper Bay and Long Island Sound—part of the time the water in the Upper Bay is higher than in the Sound and part of the time it is lower. A simple system of slope* lines, found by plotting to suitable scale the simultaneous heights of the water levels above mean sea level at the two ends of East River for each hour of the tidal cycle, will indicate quite conclusively that the tidal movement in the river is primarily hydraulic, since the range of the tide, and the high and low-water intervals, as scaled from these slope lines, agree very closely with actual observations at any place in the river. Of course this simple slope line will furnish only an approximation to the tidal conditions since it is based on the assumption that the river channel runs straight through its length with unvarying width and depth and does not take into consideration the varying depth, the changes in direction, nor the effect of fresh water entering the East River from the Hudson by way of the Harlem. Deception Pass, Washington, and Seymour Narrows, British Columbia, are other examples of this kind of tide with hydraulic characteristics.

The difficulties that often arise in connection with the harmonic prediction of estuary tides are due principally to hydraulic elements—to freshet conditions, to the restriction of the channel at low water, to the effect of the non-tidal current on the propagation of the tide wave, and to bottom and lateral friction. The Canadians have overcome the difficulty to a considerable extent in the St. Lawrence Estuary. Correct tide tables are unusually important for Quebec, because the tides at up-river points are derived from them. The tides are predicted by the harmonic analysis for Father Point, which is near the ocean and which, therefore, has a tide answering well to the harmonic treatment; for the predictions at Quebec variable empirical corrections are applied to these Father Point predictions, varying for the synodic, the anomalistic, and the declinational months. By this method the average error in the predictions, as compared with observations, has been lowered from 26 min. by the harmonic method to 6 min. by this combined method.

The tide curves for Albany, N. Y. (Fig. 11), illustrate the difficulty in making tidal predictions for upper reaches of rivers subject to freshet conditions and the necessity for long series of observations to obtain final values.

In short straits, such as the East River, New York, the harmonic analysis answers well for predictions, although the characteristics of the tides in such bodies are largely hydraulic. The hydraulic elements are brought about by periodic tidal movement in the bodies which the strait connects, and the tidal conditions, therefore, may be predicted with the same ease as the tidal movements in the main bodies.

Tide Reductions

The methods used in reducing tidal observations may be grouped under two heads: Harmonic and non-harmonic. The harmonic method's principal

* U. S. Coast and Geodetic Survey, *Special Publication No. 111*, p. 77.

use is for tidal predictions; by this method the tide wave is decomposed analytically into a number of constituent simple harmonic waves, the periods of which have astronomical definitions, while the amplitudes and phases are derived from observation. The non-harmonic is essentially a statistical method, by which the tides are tabulated for any desired argument. These reductions are made in order that the data may be in convenient form for practical use in chart-making, surveying, and general engineering work; for the prediction of tides they also include the determination of the harmonic and non-harmonic constants.

After tabulation, the high and low waters and the hourly heights are treated in a separate manner for reduction, that for the hourly heights being known as the harmonic, and that of the high and low waters as the non-harmonic. The harmonic method of reduction employing the highly refined mathematical process of the harmonic analysis, is used principally for the determination of harmonic constants for the prediction of tides for the tide tables. It is of little use in practical engineering.

At any place the times of high and low waters have approximately fixed relations to the time of transit of the moon. The relation for high waters is called the high-water lunitidal interval, or the establishment of the port, and that for low water, the low water lunitidal interval. The intervals for each place are determined by comparison of the times of the tabulated high and low waters with the times of the corresponding moon's transits.

The datums, mean high and mean low waters, are also obtained by averaging all the tabulated heights of high and low waters. The difference gives the mean range of tide at the station, and the half-sum, the half-tide level. For stations on the Pacific Coast, because of the large diurnal inequality in the heights of the high and low waters, the average of the higher high and lower low waters are also obtained, the results being the datum plane of higher high water and lower low water.

In like manner all other planes and data may be determined, such as spring range, neap range, perigean range, apogean range, mean duration of rise, mean rise interval, mean high water diurnal inequality, mean low water diurnal inequality, mean great diurnal range, etc.*

Currents in General

To the engineer, particularly the military engineer detailed to harbor and river development, the subject of currents is of equal or greater importance than that of tides. In general, currents are caused directly by tidal action, so that a clear understanding of such action is essential to a knowledge of tidal flow. They are intimately related phenomena, or may be considered as different phases of the same phenomenon, since, accompanying the vertical movement of the water caused by the tidal forces of moon and sun, a horizontal movement also takes place, giving rise to currents.

In addition to those currents due to astronomical forces working through the tides, others are brought about by meteorological conditions and physical

* The reduction of tidal data and explanations of the forms used are given in detail in U. S. Coast and Geodetic Survey *Special Publication No. 26*, "General Instructions for the Field Work of the U. S. Coast and Geodetic Survey," pp. 128–150.

differences in sea water in different parts of the oceans. These astronomical forces, combined with topographical features, as in the case of the tides, give rise to five types of currents:

(1) The rectilinear or reversing type, illustrated by the current in most inland bodies of water, such as the Hudson River, Chesapeake Bay, Delaware Bay, etc.;

(2) The so-called hydraulic type, illustrated by the currents in straits connecting two independently tided bodies of water; for example, the East River, New York, Deception Pass, Washington, and Seymour Narrows, British Columbia;

(3) The rotary type, illustrated by the currents in the open ocean and along the sea coast;

(4) The wind-driven currents of a temporary nature, produced by the friction of local winds on the surface of the water; and

(5) The permanent currents comprising the main oceanic circulation, illustrated by the Gulf Stream of the Atlantic and the Kuro Siwo of the Pacific.

The first three types are of tidal origin and are, therefore, periodic, while the last two are non-periodic and are due to meteorological conditions, dominant winds, variations in barometric pressures over the oceans, and differences in temperatures and densities of the sea water in different parts of the oceans.

Rectilinear or Reversing Type Currents

The progression or rate of advance of a tidal wave and the velocity of the current should not be confused. The velocity of the current is the actual speed of a moving particle. On the other hand the progression or rate of advance of a tidal wave is the forward motion of a shape or wave motion. Usually this is many times greater than the velocity of the current.

Except possibly in shoal channels of varying widths the rate of advance, r, of the tide wave of the progressive type is, approximately, $r = \sqrt{g\,h}$, in which h is the depth of the body of water. The velocity of the current, however, or the actual speed with which the particles of water are moving, depends on the volume that must pass a given point and the cross-section of the channel at that point.

In a rectilinear current the distance traveled by the water particles or by a free float during a full tidal cycle may be found approximately by the following equation:

(Velocity of current at strength, in knots) $\times$ (0.6366) $\times$ (duration of period of cycle, in hours and tenths) = distance, in nautical miles, floating object will be carried.*

The form of current known as the rectilinear or reversing type occurs in all inland bodies of water connected with the ocean. These currents come to a slack and reverse, the direction being designated "flood" when referring to the horizontal movement of the water in the ocean toward the land (or "up

* For greater detail, see U. S. Coast and Geodetic Survey *Special Publication No. 111*, p. 18.

stream" in inland waters), and "ebb" when referring to the movement in the ocean away from the land (or "down stream" in inland waters).

This current is intimately related to the tidal regimen of the body of water. In a body in which the tides are due to a progressive wave the time of the reversal of the current (slack-water) occurs at about half-tide, and the strength of flood and ebb at about the time of high and low water, respectively; in a body with a stationary tidal wave, slack-water occurs at about the time of high and low water, and the greatest current velocity at about the time of half-tide.

This time relation for a current station in the Hudson River, which has a progressive tidal wave, is illustrated in Fig. 25. The upper diagram is a graph of current observations on July 24, 1922, at a station in the entrance to the Hudson River. The lower diagram shows the tidal curve at the Fort Hamilton Tide Station for the same date. In rectilinear currents it is evident that any shifting up or down of the zero current line, resulting from a steady non-tidal current, would affect the periods of duration of the flood and ebb, and make them of unequal length. Such a shift would also change the times of slack-water, but would leave unchanged the times of flood and ebb strength.*

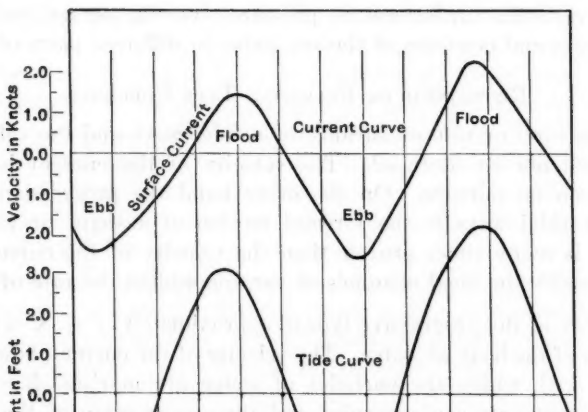


FIG. 25.—COMPARISON OF TIDE AND CURRENT CURVES, NEW YORK HARBOR.

An added advantage of such a graph for the interpretation of current data is the facility with which the duration of slack-water may be determined graphically; that is, the period during which the current velocity is less than 0.1 knot, which is becoming to mean "slack-water", although theoretically the time of slack-water is only an instant.

* For more detail, see U. S. Coast and Geodetic Survey *Special Publication No. 111*, p. 17.

The parallelism of the two curves in Fig. 25 indicates the intimate time relation between the tides at Fort Hamilton and the currents at the entrance to the Hudson; for example, the times of slack-water (upper diagram, Fig. 25), occurring at about 5:40 A. M., at 10:15 A. M., at 5:00 P. M., and at 10:45 P. M., agree very closely with the times of half-tide of the lower diagram, half-tide occurring at 4:15 A. M., 10:20 A. M., 3:40 P. M., and 11:15 P. M. This occurrence of slack-water at the time of half-tide is one of the indications that the tides of the Hudson are due to a progressive wave and not to a stationary wave.

A graph of this character has the advantage of furnishing the relation (which may be instantly detected) of the velocities of the currents at different depths. Fig. 26 is a graph of the current velocities obtained from observations at three depths, 9, 24, and 38 ft., on August 3, 1922, at a station in mid-stream, at the entrance to the Hudson.

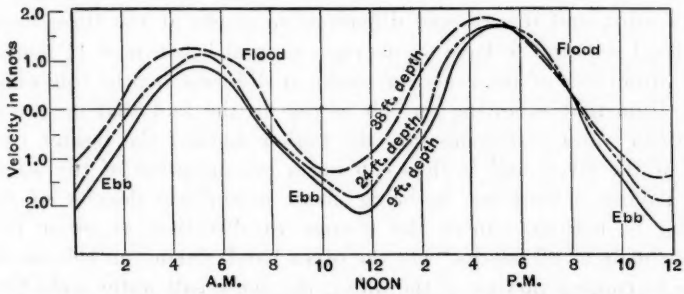


FIG. 26.—GRAPH OF CURRENT VELOCITIES AND DIRECTIONS FOR THREE DEPTHS AT STATION IN MID-STREAM AT MOUTH OF THE HUDSON RIVER, AUGUST 3, 1922.

It will be noted that the slacks from flood to ebb (between 6:00 and 8:00 A. M.) occur much more closely together than those from ebb to flood (about 2:00 A. M., and again about 2:00 P. M.), all slacks between flood and ebb, however, occurring first at the 9-ft. depth, a little later at the 24-ft. depth, and still later at the 38-ft. depth. It is again true for the afternoon slacks from flood to ebb about 8:00 P. M., except that the times are more nearly simultaneous.

An inspection of the graphs for a month of observations at this station indicate that when there is considerable diurnal inequality in the velocities of the morning and afternoon currents, the slacks of the afternoon currents from flood to ebb occur almost simultaneously, while the morning slacks at different depths from flood to ebb are separated somewhat in time; and as the diurnal inequality disappears when the moon's declination is nearing a minimum, the difference in the times of slack-water at different depths from flood to ebb become very nearly equal. The average interval between the surface and bottom slacks from flood to ebb at this station has been found from a long series to be about 12 min.

From ebb to flood, however, the times of slacks at different depths have a wider range and occur first near the bottom (1:40 A. M.), next at mid-depth (2:30 A. M.), and last near the surface (3:00 A. M.). These relations again

hold for the afternoon slack from ebb to flood—occurring first at the 38-ft. depth (1:15 P. M.), next at the 24-ft. depth (2:20 P. M.), and last at the 9-ft. depth (2:50 P. M.). Thus, the time between slack-water near the bottom and that on the surface differs by about 1½ hours. This interval has been found to average 1½ hours at this station from long series of observations.

To sum up for this station when there is diurnal inequality evident in the velocities of the current the slack-water following the weaker flood occurs first on the surface, a few minutes later at mid-depth, and last at the bottom; the slack-water following the stronger flood occurs almost simultaneously at all depths. At the time of the month when there is little diurnal inequality in the flood velocities, the slack-water following both floods occurs almost simultaneously at the three depths. The slack-water following an ebb shows very little diurnal inequality in the times of slack at all depths, occurring first at the bottom, next at mid-depth, and last on the surface. The average difference between slacks at the three different depths from flood to ebb is found to be 12 min., and the average difference in slacks at the three depths from ebb to flood is found to have an average interval between of 1½ hours.

This difference of intervals for slacks at different depths following a flood current from that following an ebb, is due to the fact that on the flood the dense ocean water is flowing into the harbor against the lighter fresh-water run-off of the river, and is thus causing a commingling of the water. This action, having a tendency to bring about a uniform density of the water from top to bottom, causes the change in direction to occur practically simultaneously at all depths. On the other hand, during an ebb current, with no force to cause a mixing of the water, the dense salt water seeks the bottom and the lighter fresh water seeks the surface. Now when the flood water first begins running into the harbor from the ocean the dense ocean water hugs the bottom and under-runs the fresh surface water coming down on the ebb. This lower layer of heavy flood water, therefore, retards and brings to a slack, first, the lower layer of ebb current, gradually gaining sufficient momentum finally to offset the inertia of the over-running, less dense surface water.

Hydraulic Type Currents

The so-called hydraulic type currents are those occurring in straits and canals, connecting two independently tided bodies of water. They are reversing currents, although they are due primarily, not to tidal action of the purely stationary or progressive type, but rather to a temporary difference in head between the two bodies brought about by tidal action. The currents in the East River, New York, and in Cape Cod Canal, are examples of this form, and a brief description of those of the East River will serve to illustrate the type in general. Although these currents are due primarily to the hydraulic gradient, the indications are that some tidal propagation is evident; but, in general, its effects may be neglected. For purposes of predicting, for example, the time of slack-water and the velocity of the currents in such straits and canals for current tables, satisfactory results are obtained by considering only the hydraulic gradient.

The East River connects Long Island Sound with New York Bay, and the tides in these two bodies of water are decidedly different, the tides in Long Island Sound being due to a stationary wave, and those in New York Bay to a progressive wave. The tides in Long Island Sound, therefore, increase in range from the mouth toward the head, from 2.0 ft. at Montauk Point (east end) to 7.2 ft. at Willets Point, at the west end of the Sound; the tide in New York Bay, however, decreases in range from 4.7 ft. at Sandy Hook to 4.4 ft. at Governors Island. This difference in range of tide at the two ends of the East River is sufficient in itself to produce a hydraulic gradient; but, in addition, the time of the tide at Willets Point at the north entrance to the East River is 3 hours or more later than the time of the tide at Governors Island. This difference in time is the larger factor in producing the temporary difference in hydraulic head to which the considerable velocity of current in the East River is due.

As examples, high water occurs at Willets Point when the tide is about at mean sea level at Governors Island, and, therefore, the current at this time sets to the southward through the East River; and at the time of high water at Governors Island the tide at Willets Point is about mean sea level and the current then sets toward Long Island Sound. This difference of tidal conditions in Long Island Sound and New York Bay, at times causing a hydraulic gradient of 4 ft. in the distance of 17 miles from Willets Point to Governors Island, gives rise in Hell Gate to currents which attain a mean velocity, at strength, of about 5 knots; and occasionally currents have a velocity of 8 knots.

While it is a comparatively simple matter to predict the time that the heads at the two ends will be equalized and that slack-water should occur, account must be taken, with conditions such as these, of the momentum of this mass of water and of the time of the continuance of the flow after equalization of head. This information can be obtained only from observations, and to obtain mean conditions these observations should be continued over a considerable period, since the range of the tide between springs and neaps varies more than 20%, and, therefore, the current velocity at the different phases of the moon varies proportionately.

If a long series of tidal observations at a suitable place near-by are available, short series of current observations may be reduced to mean values by comparison with these tide observations, since the currents are intimately associated with the tidal regimen of the bodies of water and vary during the tidal month proportionately with the range of tide. The procedure is explained hereafter under "Current Reductions".

The hydrodynamic problems that arise in a canal connecting two large bodies of tidal waters are of so involved a nature that even where solutions have been found they necessitate a great deal of complicated numerical work. Since, however, the engineer is interested chiefly in the maximum velocities, or those that occur at times of strength of the current, simpler formulas that will give approximate results are used.

Ocean Currents

The along-shore currents of the coast have considerable importance in coast erosion and accretion studies, and since these are intimately associated with the general ocean circulation, engineers should have a general knowledge of this circulation.

Little of a quantitative nature has been done in the study of oceanic circulation by direct observations of currents. The set and drift of ocean currents have been mapped in a general way; but the principal information on which these results are based is of a qualitative nature obtained from a record of the drift of bottles and of wrecks, and from a difference between the true and the dead reckoning positions of vessels, the determination of which does not permit of a high degree of accuracy. All these, however, furnish qualitative data and quantitative data can come only from systematic observations either directly of the current or of densities and temperatures, all of which may be considered an international, rather than a national, problem.

Three reasons may be assigned for this near neglect of a phenomenon so closely associated with the subject of tides: Obviously, tidal observations may be made more economically than current observations, for which expensive vessels with full complement of officers and crew are necessary. Secondly, associated with periodic tidal currents are non-tidal currents which, in general, are produced by the wind and by differences in temperatures and densities of sea water. Data for predictions of tidal currents, since they are periodic, may be obtained from observations extending over a comparatively short time, while for non-periodic non-tidal currents a much longer series of observations is required to furnish average conditions and to permit of predictions of any degree of accuracy. Another reason is that the oceans may be considered the highways of commerce for all nations, and, therefore, the study of its problems an international rather than a national undertaking.

The subject of ocean currents is now receiving the attention of physical oceanographers, particularly in the North European countries in their fisheries conservation studies—Bjerknes, Helland-Hansen, Nansen, and Pettersson, in Europe, and McEwen, of the Scripps Institution of Oceanography, in America.

Dynamic oceanography provides a method for mapping currents over extensive ocean surfaces, and it is by these methods, rather than by direct current observations, that the present meager knowledge of ocean currents will undoubtedly be supplemented. Given the temperature and salinity observations from several known depths at stations systematically located over an ocean area, then the direction and velocity of the current are readily predicted and mapped by the physical oceanographer. Studies made at Bergen, Norway, covering density and temperature observations obtained by the International Ice Patrol, indicate the probability that, from a sufficient number of observations of density and temperature over an area, current charts can be prepared to show the probable direction and velocity of the relatively deep-seated ocean currents several weeks in advance.

The principal maritime nations have made current observations, however, on their inland waters and along their coasts. Within the last few years the U. S. Coast and Geodetic Survey has given attention to the study of coastal currents, both tidal and wind-driven. Recently, current tables have been issued containing data of importance on this subject which are vital to the safeguarding of life and property in coastwise shipping. This is particularly true on the Pacific Coast in that its currents, like its tides are more complex than those on the Atlantic; and, in addition, in that its coast harbors are many miles apart, the sailing courses long, and the periods of foggy and stormy weather of comparatively frequent occurrence.

Although the total coastal current is made up of a combination of two components, the periodic, or tidal, current and the non-periodic, or wind-driven, current, analysis of a long series of current observations made at any station serves to separate the two, so that the proper correlation can be made between each component and its causes. That is, the velocity and direction of the tidal current may be referred to the predictions at some standard port, and the velocity and direction of the wind-driven current correlated with the force and direction of the local wind prevailing at the time. These two coastal currents, although combined, are of distinct characters and will be taken up separately.

Coastal Tidal Currents.—The current observations, on which these studies of coastal tidal currents have been made, have been obtained on the light vessels along the coasts, through the co-operation of the Lighthouse Service. While the data obtained relate chiefly to the vicinities of these vessels, general laws of the action of these currents can be deduced, which, usually, will apply to other sections of the coast.

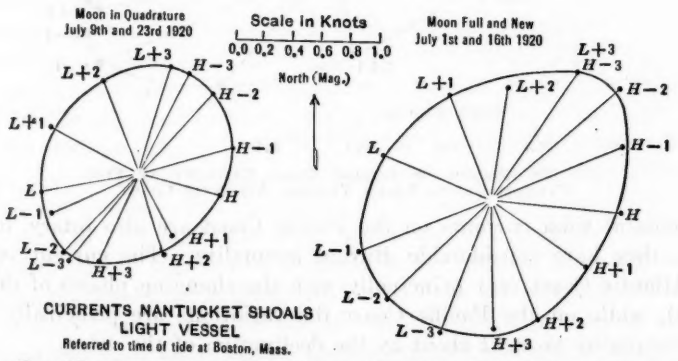


FIG. 27.—GRAPH OF ROTARY TIDAL CURRENTS, NANTUCKET SHOALS LIGHT VESSEL, ATLANTIC COAST, AT TIMES OF DIFFERENT PHASES OF THE MOON.

On the open coast the tidal current is not of the rectilinear or reversing type with its period of slack-water; nor is the current ever slack—its direction changes constantly in a rotary movement. The varying velocities throughout a tidal cycle when plotted to scale on polar-co-ordinate paper approximate an elliptical shape illustrated by Fig. 27. This shows velocities and directions

of current at Nantucket Shoals Light Vessel referred to the time of tide at Boston. For example, from the left diagram at $H-2$, that is, 2 hours before high water at Boston, the current sets about south by east with a velocity of 0.6 knots.

It should be noted that the current is constantly changing direction clockwise at the rate of about 15° per hour from H (high water) through each hour to L (low water) and so on to H again; and that the velocities are such that a curve through the extremities of ordinates approximates an ellipse. The velocity of the tidal current along the Atlantic Coast varies with the times of neap and spring tides (compare the right and left diagrams of Fig. 27). The direction of the set of the current, however, is in general the same at any time of the month for the same tidal hour at a standard port; for example, in both diagrams, H is in the same general direction from the origin; at L-3 (3 hours before low water at Boston) the direction is the same for both diagrams, and likewise for each tidal hour.

A graph of the tidal current at Five Fathom Bank Light Vessel (Fig. 28) illustrates that toward the coast tidal currents assume more nearly the character of the rectilinear or reversing type, the ellipse becoming more elongated than those for Nantucket Shoals Light Vessel, which is about 40 miles offshore.

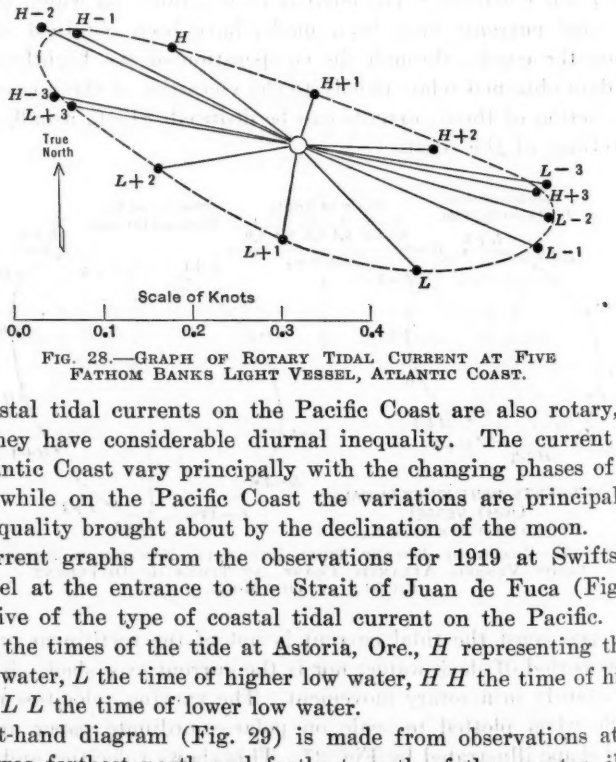


FIG. 28.—GRAPH OF ROTARY TIDAL CURRENT AT FIVE FATHOM BANKS LIGHT VESSEL, ATLANTIC COAST.

The coastal tidal currents on the Pacific Coast are also rotary, but, like its tides, they have considerable diurnal inequality. The current velocities on the Atlantic Coast vary principally with the changing phases of the moon (Fig. 28), while on the Pacific Coast the variations are principally due to diurnal inequality brought about by the declination of the moon.

The current graphs from the observations for 1919 at Swiftsure Bank Light Vessel at the entrance to the Strait of Juan de Fuca (Fig. 29), are representative of the type of coastal tidal current on the Pacific. They are referred to the times of the tide at Astoria, Ore., H representing the time of lower high water, L the time of higher low water, $H\,H$ the time of higher high water, and $L\,L$ the time of lower low water.

The left-hand diagram (Fig. 29) is made from observations at the time the moon was farthest north and farthest south of the equator, illustrating

the variation in velocity due to diurnal inequality; that is, at the time the daily tidal forces are at a maximum, when the moon is at its greatest declination. It will be seen that on the Pacific the velocity of the coastal tidal current, like the range of the tide, is greater during one tidal cycle than during the next. For example, one hour after lower high water at Astoria the current at Swiftsure Bank Light Vessel has a velocity of about 1 knot; whereas one hour after higher high water the current has a velocity of only about ¼ knot. On the Atlantic Coast the two ellipses of a tidal day are practically the same, whereas the ellipse through one tidal cycle on the Pacific Coast is considerably different from that of the next tidal cycle.

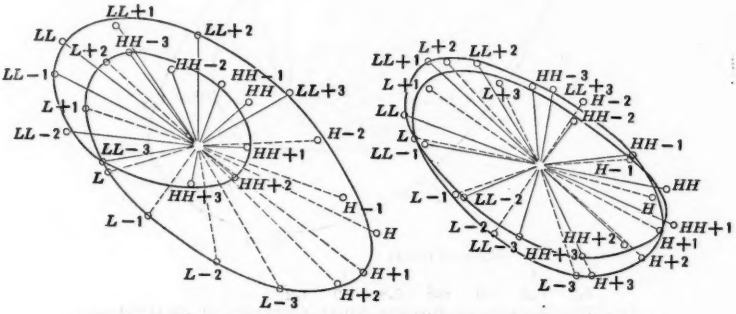


FIG. 29.—GRAPHS OF ROTARY TIDAL CURRENT, SWIFTSURE BANK LIGHT VESSEL, PACIFIC COAST, AT TIMES OF DIFFERENT DECLINATION OF THE MOON.

The right-hand diagram, Fig. 29, is a graph of a current at Swiftsure Bank Light Vessel at the time the moon is nearing the equator; that is, when the moon has its least declination. The semi-daily forces at this time predominate as in the tides, and the diurnal inequality practically disappears from the current graph as it does in a tide curve at this time.

The permanent set of the current at a station due to causes not tidal may be readily obtained by this method of plotting the tidal current graph. In Fig. 30 the velocities and directions of the tidal current, as averaged for the particular period desired, are plotted from a common origin (focus of the rays) and the true center of the ellipse, *C*, is then determined. The direction and distance to scale of *C* from the origin represents the permanent set of the current; that is, superimposed on the tidal currents there was a constant set during the month of August, 1919, passing Swiftsure Light Vessel at a rate of almost ½ knot in a direction N. 40° W. This permanent set may be ascribed to the fresh-water run-off from the Strait of Juan de Fuca, the light vessel being stationed just off the entrance to the Strait; it may be due to several causes, among which are prevailing winds, fresh-water discharge from a strait or river, and permanent sets like the Gulf Stream.

Since the rotary tidal currents along the open coast are so closely related to tidal action, it is evident that those components due to tidal action alone may be readily and accurately predicted as to their velocities and directions by comparison with the times of high or low water at some standard port.

The problem, however, is not so easily solved, for this tidal current is only one of the components of the total coastal current. The other, due to local winds, is not only non-periodic, defying accurate predictions a long time in advance, but, in general, is of greater velocity than the tidal currents, and, therefore, proportionately of greater importance.

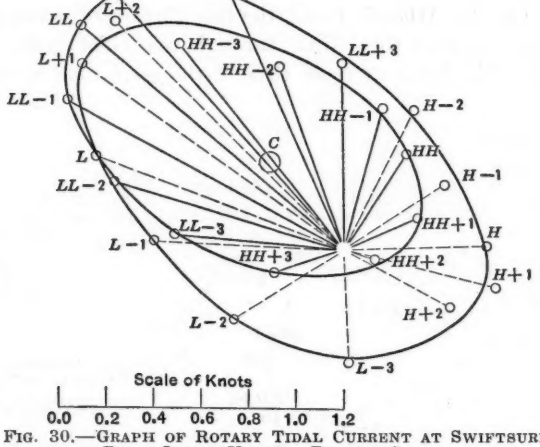


FIG. 30.—GRAPH OF ROTARY TIDAL CURRENT AT SWIFTSURE BANK LIGHT VESSEL, PACIFIC COAST.

Coastal Wind Currents.—It has been brought out that the rotary tidal-current ellipse may be displaced from its origin by a permanent set due to fresh-water run-off; at times, the current produced by heavy winds, however, retards the regular tidal current to such an extent that the origin is displaced considerably outside the ellipse. At Frying Pan Shoals Light Vessel from observations made during a northeast storm from January 29 to February 2, 1920 (Fig. 31), the wind velocity varied during the five days from 15 to 60 miles per hour, averaging 34.6 miles; the wind direction varied from north-northeast to northeast.

The origin of the graph is at P. The direction of the current varied throughout the tidal cycle from south to west and back to south; that is, from $H+1$ to $L+3$ by way of $L-2$, and then from $L+3$ to $H+1$ again by way of $H-1$. The velocities also varied because of the tidal forces throughout a tidal cycle and the curve through the extremities of the velocity ordinates approximates an ellipse; that is, the current, although setting to the southwestward during the entire duration of the storm varied in velocity by the amount of the acceleration and retardation due to the tidal component.

In an ocean of infinite depth it may be shown* from theoretical considerations that a current produced by winds should be deflected on the surface 45° to the right of the wind in the Northern Hemisphere and 45° to the left in the Southern Hemisphere, due to the deflecting forces of the earth's rotation.

* "On the Influence of the Earth's Rotation on Ocean Currents," *Arkiv for Mathematik, Astronomie, och Fysik*, Vol. 2, No. 11, 1905.

Near the coast it is to be expected that this deflection should be considerably modified by the configuration of the shore. Recent investigations from the comparatively long series of continuous current observations obtained during the past few years at different light vessels have verified this theory and brought out the important fact that, contrary to the belief of the mariner, a local wind creates a current setting not in its own direction, but in a direction about 20° to the right of the wind.

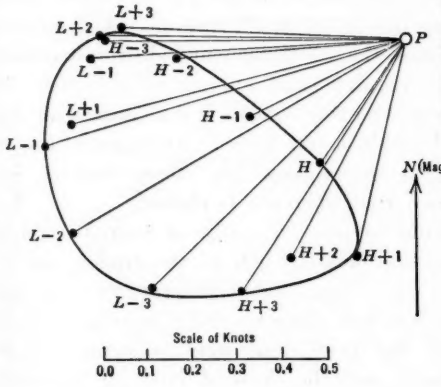


FIG. 31.—GRAPH OF CURRENTS AT FRYING PAN SHOALS LIGHT VESSEL, ATLANTIC COAST, FROM OBSERVATIONS MADE DURING NORTHEAST STORM, JANUARY 29 TO FEBRUARY 2, 1920.

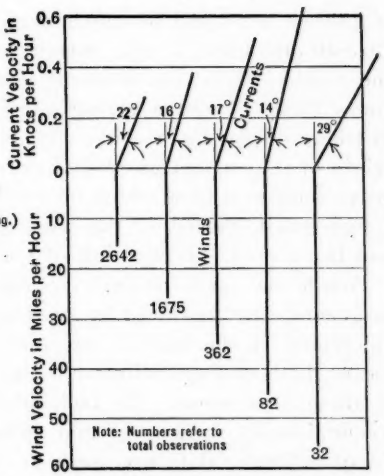


FIG. 32.—RESULTS OF OBSERVATIONS OF WIND-DRIVEN CURRENTS, DUE TO NORTH-NORTHWEST WINDS AT BLUNTS REEF LIGHT VESSEL, PACIFIC COAST.

Since a large number of observations of this character are necessary for averages of any degree of accuracy, and also for the elimination of tidal current in the resulting data, it is evident that observations of wind-driven currents have to be made over a considerable time in order to get a number of winds of 10 miles per hour, 20 miles per hour, etc., blowing from various directions. In Fig. 32 are represented diagrammatically the results from a number of observations over a period of two years (1919-20) for a north-northwest wind at Blunts Reef Light Vessel, Pacific Coast, for velocities from 10 to 60 miles per hour. The deflections of the slanting lines from the vertical indicate the deflections of the resulting currents to the right of the wind direction.

While the general law applies to the average current produced by any given wind velocity, observations are necessary at intervals along the coast, particularly well inshore, to determine the actual velocities for that particular locality. For coastwise navigation, however, the general law will hold with sufficient accuracy for practical results within 5 to 20 miles offshore; it has been found that the velocity varies fairly proportionately with the wind velocity and is about 2% in knots of the wind velocity in miles per hour on the Pacific Coast and 1½% on the Atlantic Coast.

In practice, it is necessary to take into consideration the combination of this wind-driven current with the periodic tidal current, the tidal current being obtained from predictions and the wind-driven current estimated from the local wind at the time. In actual practice the resultant current may be readily obtained from a diagram drawn to suitable scale showing the velocity and direction of each component, tidal and wind-driven, the resultant direction and velocity being scaled from this graph.

At times, however, wind-driven currents of considerable velocity appear to precede the wind to which they are due. The writer during two winters, 1915-16 and 1916-17, took current observations from the Carolinas well down the Florida Coast and from the shore out to the Gulf Stream. A number of times these showed a strong set from the southwestward during moderate weather, displacing considerably the regular tidal current ellipse from the origin of the current graphs. In every case this condition was indicative of a heavy southwest blow which followed within a few hours. At times, of course, a long swell, preceding the wind, may accompany this strong current, but sometimes even this warning of treacherous currents is absent.

While the main oceanic circulation involves a study of currents which, in general, are produced by predominant winds, such as the trades, and by differences in barometric pressures, densities, and temperatures over the oceans, little of a quantitative nature has been done toward a systematic study of these phenomena. In fact, what has been done may be considered as reconnaissance work only and volumes may be devoted later to obtaining the quantitative data necessary to such a study.

Reduction of Currents

Mean Values.—Since the tidal currents in bays and estuaries are so intimately connected with the tidal regimen in these bodies, a short series of current observations may be reduced to mean values by means of tidal data.

In the non-harmonic analysis of current observations the times of slack-water and greatest velocity are referred to the times of high and low water of the tide at some suitable place near-by. In this method the time of current is reduced to approximately mean values, since the changes in the time of current approximate the changes in the time of the tide. It is necessary, however, to reduce the velocities to mean values.

The velocities of a short series of ordinary current observations of the rectilinear or reversing type, caused by a progressive or stationary wave, may be reduced to mean values by multiplying by a factor equal to the reciprocal of the range of the tide for the same period of time during which the current observations were made, divided by the mean range of the tide for the place. The velocities of hydraulic currents, however, vary as the square root of the head (that is, of the difference in the height of water at the two ends of the strait); and to reduce the velocities of a short series of hydraulic type currents to mean values, it is necessary to multiply by a factor which is the square root of that used for ordinary tidal currents referred to heretofore.

Before multiplying the velocities of currents of both types by the suitable factors to reduce them to mean values, the non-tidal current, due to fresh-

water run-off, must be eliminated. This may be done by taking as the tidal current the half sum of the observed flood and ebb strengths, and this half sum is multiplied by the correction factor in each case.

The relations as to time existing between currents and tide apply as to the moon's changing phases and also as to the moon's changing distance from the earth; but as to the moon's changing declination, the current and tide do not respond alike, the diurnal variation in the tide at a place usually being greater than the diurnal variation in the current. In regions where the tide has considerable diurnal inequality, as on the Pacific Coast of the United States, the time relations between tide and current are subject to variations. In such cases, however, it is possible to refer the current at any place to the tide at some other place that has a comparable diurnal inequality.

Instruments for Current Observations

The instruments made in the United States for the observation of currents have been devised principally for shallow depths and for use in rivers. Little attention has been given, therefore, to attachments for furnishing the direction of the current.

There are a number of types of meters, described in the catalogues of American instrument manufacturers; practically all, however, are made on a principle somewhat similar to the anemometer used by the meteorologist—several metal cups on a carriage revolving in the current and making an electric contact at each revolution of the carriage. This contact, making an audible "tick", is transmitted through insulated cable and telephone receiver to an observer on a vessel.

The Price current meter (Fig. 33) illustrates this type. The meter is suspended on station by a triple-strand insulated cable, 0.133 in. in diameter, made up of three enameled wires, one of which is piano wire, 0.0335 in. in diameter, for carrying the weight of the meter, and the other two, copper wires, No. 20 B. and S. gauge, for the electric circuit to the telephone receivers on board the observing unit.

To hold the meter from a davit of the observing vessel vertically on station in strong currents, a spherical 200-lb. cast-iron weight is suspended near the bottom on a ½-in. steel wire cable, and the meter lowered and raised along this cable by means of its own suspending cable and two small shackles (Fig. 33). The meter is rated before and after a survey lasting several months, so that the revolutions may be reduced to exact velocity.

Since the meters furnished by manufacturers in this country do not have a means for determining the direction of sub-surface currents—an important matter in current surveys for local peculiarities of currents in large ports off piers and slips—the U. S. Coast and Geodetic Survey has developed a device for this purpose. It is known as a "Bifilar Suspension Current Direction Indicator", and appears to have solved in a simple and inexpensive manner the problem of the determination of the direction of these currents in the smooth waters of a harbor. The determinations, however, are vitiated in rough water, where the rolling of the vessel causes the comparatively heavy device to swing.

Briefly, the device consists of a pipe, with rudder attached, free to turn in azimuth by virtue of a ball-bearing joint at its connection with an outrigger or davit on a vessel. The pipe is lowered to any moderate depth desired. A graduated pelorus at the davit indicates by a pointer the direction the bar is forced to take by virtue of the rudder acting through the taut aircraft wires.

A later development of the device (Fig. 35) permits not only of simultaneous observations at three depths, but also lessens the labor of operation by obviating the necessity of continually lowering and raising a single rudder for observations at various depths.

Foreign instrument makers have given consideration to attachments within the instrument for the determination of direction. Among others are the Ekman meter* and the Pettersson meter.† The former accomplishes the purpose by allowing metal balls, the size of ordinary shot, to drop into different compartments, one for each of the principal points of the compass, as the meter swings in azimuth; the latter by photographing on a narrow, sensitized film the graduated periphery of a glass disk, which is held in azimuth by a magnetized steel bar.

The Pettersson type (Figs. 34 and 36) is a self-recording meter and makes a photographic record each half hour of the total current passing the meter during the preceding half hour, and, in addition, of the magnetic direction of the current. The metal cartridge (left, Fig. 34), containing the recording device, is inserted into the heavy brass cylinder (right, Fig. 34), and the cap screwed down in place on top of the cylinder by heavy lug screws. A rubber gasket in the face of this cap prevents any water entering even at pressures of considerable depth.

The whole apparatus is supported on a ball-bearing shackle, permitting of a free movement in azimuth. The vane or rudder to the right (Fig. 34) forces the meter to head into the current.

The details of its operation are shown in Fig. 36. There is no mechanical connection between the current vanes, W W, which are wholly outside the cylinder, and the recording device on the inside of the cylinder. The vanes, W W, through a train of gears, turns a permanent magnet, M_1. This magnet in turn causes Magnet M_2, on the inside of the cylinder, to rotate in synchronism; Magnet M_2, through a train of gears and vertical shaft, causes the glass disk, D_1, to revolve. This glass disk is graduated on its periphery and the number of graduations past a given point in an half hour gives the number of revolutions made by the vanes, W W, and thus the current velocity, during that half hour. The glass disk, D_2, graduated on its periphery to every 10°, carries on its surface a magnetized needle, which holds it in azimuth.

The clock-work causes a sensitized photographic film to move at a uniform rate past an opening just above the two glass disks, D_1 and D_2. The clock-work also causes a light, L_1, to flash each half hour, thus making a record on the film of the graduations on both disks opposite the initial line, across the opening, at that instant. B_1 and B_2 are two ordinary dry cells and J is of

* "A Text Book of Oceanography," by T. T. Jenkins, p. 133, E. P. Dutton, N. Y.

† "A Recording Current Meter for Deep Sea Work," by Dr. Hans Pettersson, *Quarterly Journal*, Royal Meteorological Soc., Vol. XLI, No. 173, January, 1915.

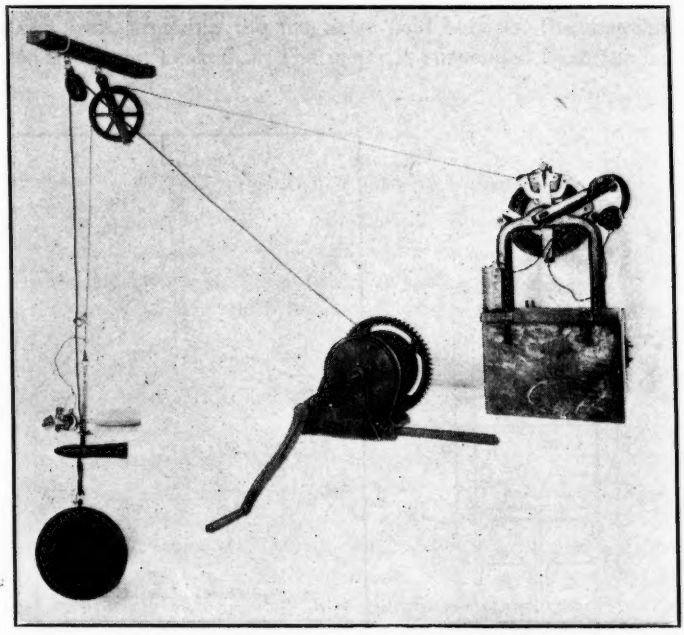


FIG. 33.—DETAILS OF INSTALLATION ON THE CURRENT SURVEY VESSEL OF THE PRICE CURRENT METER.

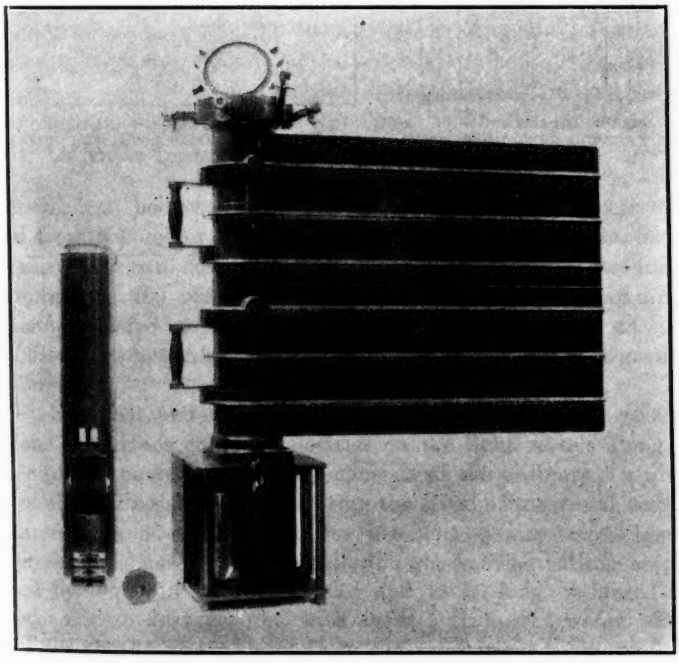


FIG. 34.—VIEW OF THE PETTERSSON SELF-RECORDING CURRENT METER.



Swedish soft iron, breaking the magnetic field between the magnets and the magnetized needle on Disk D_2. The meter is suspended from the ball-bearing shackle, S.

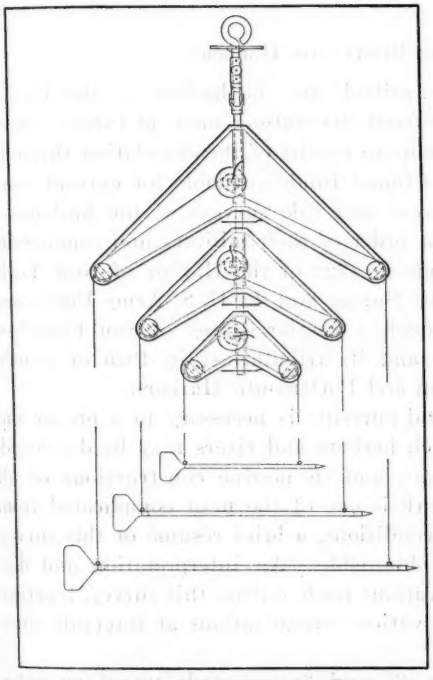


FIG. 35—BIFILAR SUSPENSION CURRENT DIRECTION INDICATOR, GIVING RECORDS AT THREE DEPTHS.

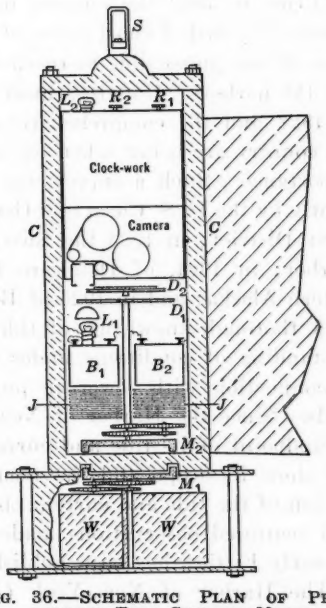


FIG. 36.—SCHEMATIC PLAN OF PETTERSSON TYPE CURRENT METER.

The meter can be lowered to any reasonable depth and left on station, suspended from a buoy, or raft, requiring no attention until the film has run out and the clock run down. The clock will run for 2 weeks and the film will accommodate 700 observations, sufficient for the corresponding record. This meter was tested by the U. S. Coast and Geodetic Survey on the current survey of Southeastern Alaska during the summer of 1925. It gave excellent satisfaction.

The U. S. Coast and Geodetic Survey has obtained good results in the observations of surface current velocities on the light vessels along the coast by the use of the current line. This consists of the ordinary log-line used in the days of the sailing ship for estimating the speed of the vessel, except that it is graduated to a 60-sec. interval for use with a stop-watch instead of a shorter interval as used with the old sand-glass. The sailing ship chip-log is replaced by a pole 3 in. in diameter and 15 ft. long, weighted with lead at the lower end to float upright, with about 1 ft. out of water. The line is graduated for the 60-sec. interval from the proportion, 3 600 sec. : 60 sec. = 6 080 ft.: X, or 101 ft. 4 in. representing 1 knot. The knot is subdivided into tenths by silk fish-line tabs.

The current line and pole furnishes a more reliable record of current velocities obtained from a vessel in rough water on the open coast than the current meter, because of the acceleration of the meter due to the rolling and pitching of the vessel.

Current and Tide Survey of Harbors

Prior to 1922 the current data available for the harbors of the United States consisted of short series of scattered observations made at various times. Thus it was impossible to reach conclusive results of the circulation throughout the harbors. Congress made additional funds available for current work in 1922 and the comprehensive current and tide surveys of the harbors of the country are being taken up in the order of their military and commercial importance. Such a survey was made in 1922 of the Harbor of New York, jointly by the U. S. Coast and Geodetic Survey and the U. S. Army Engineers, First District; in 1923 the Survey made a similar survey of San Francisco Harbor; in 1924, of Delaware Bay and its tributaries; in 1925 of Southeastern Alaska; and in 1926 of Boston and Portsmouth Harbors.

A thorough knowledge of tides and currents is necessary to a proper understanding of conditions under which harbors and rivers may be developed, in connection with sewerage problems, and in marine constructions of all kinds. Since the Harbor of New York is one of the most complicated from a standpoint of its tide and current conditions, a brief résumé of this survey will show its scope and the results obtainable. An interpretation and discussion of the tide and current observations made during this survey, together with scattered observations made by various organizations at intervals since the early Fifties, has been published.*

The Harbor of New York (Figs. 37 and 38) is made up of an outer bay, a narrows (at Fort Hamilton), an inner bay, a tidal river, a sound, and two straits, connecting independently tided bodies of water. Since the tidal and current conditions in all sections of the harbor are so intimately and vitally connected, continuous simultaneous observations throughout the entire harbor, of course, would make for an ideal survey. Unquestionably, it was uneconomical and impracticable even to approach this ideal and the problem was met by dividing the harbor for the purpose of the survey into different major sections and linking together the observations at one or two principal stations of one section while observations were in progress in the contiguous section. The actual observations continued day and night for about three months.

The dense traffic in the harbor, particularly in the East River and in the Lower Hudson, presented a rather troublesome question. The constant danger of collision to the observing units of the Survey, particularly at night and during thick weather, made it advisable in the parts of the harbor where the traffic was most dense, to utilize large flat-bottomed barges instead of the light 60-ft. launches generally used on this class of work.

The times of slack-water are of importance to shipping in a harbor in which the current velocity is considerable and the vessels entering the harbor,

* U. S. Coast and Geodetic Survey *Special Publication No. 111*, "Tides and Currents in New York Harbor"

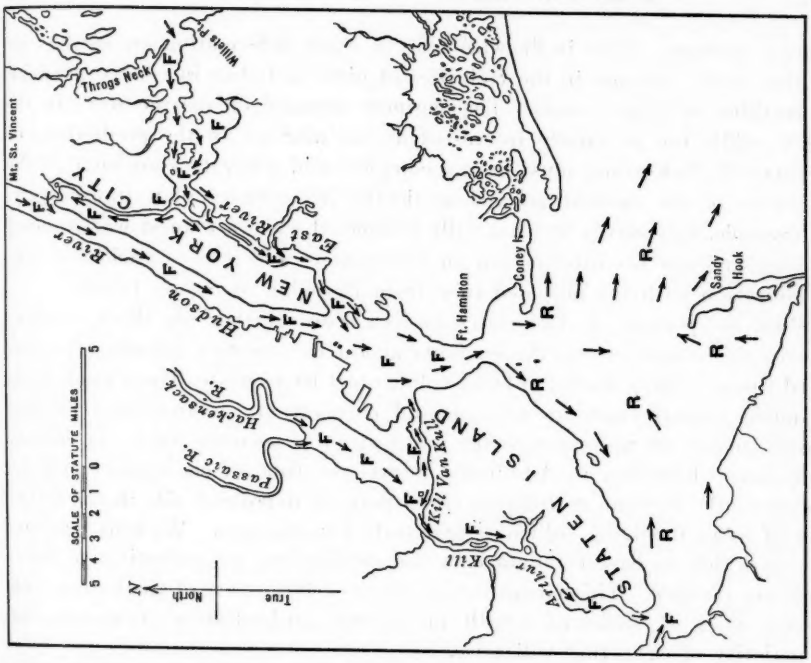


FIG. 38.—TIDAL MOVEMENT IN NEW YORK HARBOR AT TIME OF LOW WATER AT FORT HAMILTON, N. Y. F INDICATES FALLING TIDE.

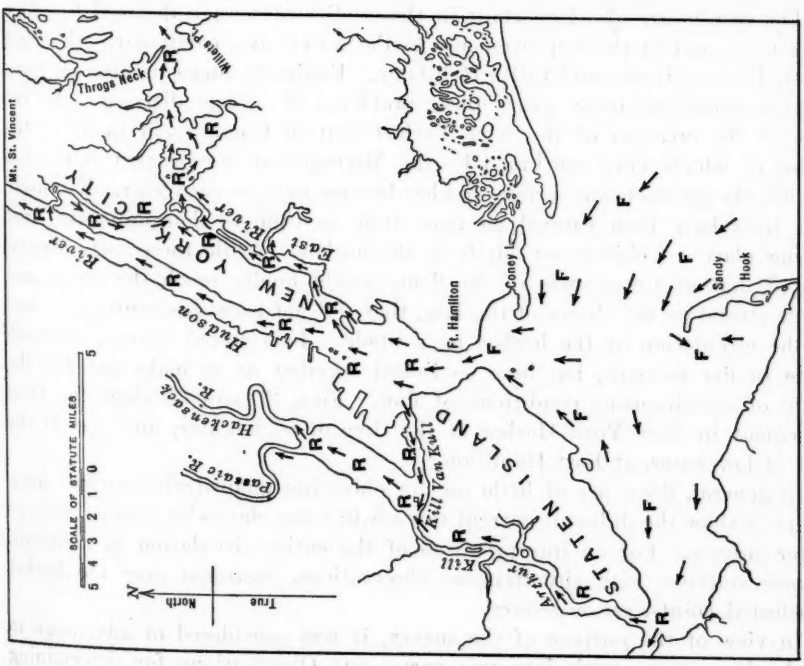


FIG. 37.—TIDAL MOVEMENT IN NEW YORK HARBOR AT TIME OF HIGH WATER AT FORT HAMILTON, N. Y. R INDICATES RISING TIDE.

of large tonnage. This is likewise true of exact information on local peculiarities of the currents in the vicinities of piers and slips in connection with the berthing of large vessels. The engineer engaged on marine work in the harbor, while not so vitally interested as the mariner in the predictions of the times of slack-water, must have a complete and accurate knowledge of the conditions of the current at various depths in order to plan intelligently. For example, this survey was not fully completed when a request was received from an engineer for information on the current velocities at various depths in connection with the proposed tube from Brooklyn to Staten Island.

Since the periods of slack-water in New York Harbor are short, particularly in those parts where the currents attain the greatest velocity, the predicted times of these slacks, to be of value, must be accurate. These predictions are made several years in advance and observations as furnished by this current survey are necessary to the preparation of accurate tables of current predictions. Likewise, in Admiralty Courts the time of slack-water and the velocity of the current at different times play an important rôle in the settlement of cases involving collisions and suits for damages. Without accurate data on which to base computations and conclusions no authoritative statements are possible. This recent survey covers a large part of the harbor with current series of sufficient length to warrant authoritative statements for the majority of cases that will arise.

The results are also important in the studies of sewage disposal for New York City, and in the improvements to the waterways, particularly the East River, Harlem River, and Little Hell Gate. Engineers and commissions have been studying for many years these problems of sewage disposal. At the time of the creation of the New York Pollution Commission in 1903, the duties of which were continued by the Metropolitan Sewerage Commission in 1906, the question was serious and has become more so each year. Extensive float tests have been carried on from time to time to determine the final resting places of objects set adrift in the harbor. While these tests demonstrated that at times some of the floats would finally reach the ocean and others strand on the shores of the Bay, they did not furnish quantitative data on the circulation of the harbor as a whole. This recent survey, although made in five sections, has been so linked together as to make possible the study of simultaneous conditions of flow. Figs. 37 and 38 show the tidal movement in New York Harbor at the time of high water, and also at the time of low water, at Fort Hamilton.*

In general, floats are of little use for obtaining quantitative current data, except to show the different current threads in some classes of studies, such as oyster culture. For an interpretation of the entire circulation in a harbor, definite stations, with simultaneous observations, scattered over the harbor at selected points, are necessary.

In view of the purpose of the survey, it was considered of advantage to divide the current work into two parts: (*a*) Observations for determining

* U. S. Coast and Geodetic Survey *Special Publication No. 111*, in which the data from this survey are discussed, contains thirteen such diagrams of the harbor as a whole, showing the generalized tidal movement throughout the harbor each hour of the tidal cycle.

circulation; and (*b*) observations for determining local conditions. The first comprised making observations as nearly as possible simultaneously throughout the harbor for the purpose of obtaining data on the full circulation of the harbor water, and after the completion of this work, re-occupying the principal stations and obtaining Data (*b*) at secondary stations off the piers and docks regarding local peculiarities of the currents.

Harbor Circulation.—The impracticability of carrying on current observations simultaneously in a harbor of such extent as that of New York is evident. The harbor, therefore, was divided into five sections for the purpose of planning and carrying on the survey for circulation. For each section the principal stations were so planned as to provide an overlapping of simultaneous observations in contiguous sections, in order to link one section to the next.

In addition to the principal stations each area was covered by a number of secondary stations of either 13 or 26 hours each. A number of these stations were arranged so as to furnish observations of three to five units at a cross-section.

Local Conditions.—The observations at the stations of the various sections completed the operations for harbor circulation. For the observations for local conditions in the vicinities of piers the harbor was divided into three sections: The East River from Governors Island to Blackwells Island, the Hudson River from the Battery to 37th Street, and the Upper Bay. During the time the current observations, both for circulation and local conditions, were in progress, observations were being made on the three light vessels stationed off the entrance to New York Harbor, Fire Island, Ambrose, and Scotland Light Vessels, and also on Cornfield Point Light Vessel in Long Island Sound.

When the survey of New York Harbor was made the Pettersson meter had not been thoroughly tested. Price current meters solely were used for obtaining current velocities and, in addition, the ordinary current lines and current poles, already described, were used for check observations of surface velocity. The "single count" contact of the Price meters gave better results than the "Pentahead" for the currents encountered.

The chemicals in the water of the harbor at times had a deleterious effect on the points of the fine contact wires of the meters, and it was necessary to replace these at intervals. A supply of this wire was furnished by the manufacturers of the meter and new contact points were made and soldered on in the field without causing delay in the observations, since a number of spare meters were kept on hand.

The observations at the current stations were made, with some exceptions, from large flat-bottomed barges or scows, which were moored from four anchors, one from each corner, to hold them exactly on station. In those parts of the harbor where traffic was not sufficiently dense to constitute a danger to the units, the observations were made from launches, 60 to 75 ft. long, moored fore and aft.

At each station the current observations were made half-hourly with the meter, and checked hourly as to surface velocities by observations with the pole and current line. The direction of the surface current was determined by laying the current line, attached to the current pole, across a graduated

brass plate secured to the taffrail of the launches, or to the deck of the barges, and referred to a compass; the direction of the sub-surface current was determined by the Bifilar suspension current indicator.

Velocities were determined with the meters by observations near the surface, at mid-depth, and near the bottom. In general, readings were taken at two-tenths, five-tenths, and eight-tenths of the depth at each station, beginning with the one near the surface, followed by the mid-depth and near-the-bottom observations. Then the order was reversed and, before the meter was drawn up, the observation near the bottom was repeated; then the mid-depth; and, finally, the one near the surface.

Special weights of 60 lb. each and special hangers were provided for the Price meters because of the strong currents. This weight, however, was not sufficient to keep the meters in a vertical position below the vessel. In an attempt to overcome this difficulty a small winch was installed on the deck of each of the observing units for carrying a ½-in. wire cable holding a 250-lb. concrete block, which was hung just clear of the bottom. The meter was lowered and raised along this cable by a small shackle secured to its hanger. Subsequent experience, however, on later surveys has demonstrated that a 200-lb. spherical cast-iron weight, with its smaller surface (Fig. 33), is the more satisfactory.

In Hell Gate the velocity of the current and the traffic prohibited the anchoring of launches or barges. To obtain observations in this important waterway one of the concrete blocks was lowered from the Hell Gate Railroad Bridge by a ¾-in. cable and the meter run up and down this cable. After a series of 6 hours' observations, a tow ran into the cable and carried away the whole apparatus. The observations were then continued for a period of 13 hours from a drillboat of the Army Engineers moored off Hallets Point. The drillboat was run down a few hours later.*

Conclusion

The foregoing furnishes a brief outline only of the subject of tides and currents. Extensive further references are available.†

The finished products of the tide and current work of the U. S. Coast and Geodetic Survey for the use of the engineer and mariner appear in several forms: In tide tables and current tables which foretell the state of the tide and current for future years; in publications on tidal bench marks, which definitely fix for the engineer the various datum planes over all the coasts of the United States; and in special publications devoted to a detailed interpretation of the tide and current phenomena of the various ports of the country.

* For more detailed discussions of the data from comprehensive tide and current surveys and of the methods and procedure used on these surveys, refer to the following publications of the U. S. Coast and Geodetic Survey: *Special Publication No. 111,* "Tides and Currents in New York Harbor"; *Special Publication No. 115,* "Tides and Currents in San Francisco Harbor"; *Special Publication No. 123,* "Tides and Currents in Delaware Bay and River"; *Special Publication No. 127,* "Tides and Currents in Southeastern Alaska"; and *Special Publication No. 124,* "Instructions for Tidal Current Surveys."

† For a more detailed, non-mathematical treatment of the phenomenon in all phases the student is referred to "The Tide," by H. A. Marmer, 1926, D. Appleton & Co., N. Y., and "Tides and Kindred Phenomena in the Solar System," by G. H. Darwin, Lond., 1911. For a mathematical treatment of the subject he should consult "Manual of Tides," by R. A. Harris, 1894–1907, Govt. Printing Office, Washington, D. C. These publications carry additional references to published material on the subject.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

NOTES ON ARCHED GRAVITY DAMS

By B. F. Jakobsen,* M. Am. Soc. C. E.

Synopsis

Formulas are developed for a gravity dam having a triangular section and a vertical up-stream face that is curved in plan to a radius, R_u. It is shown that the radius of curvature influences the stresses considerably and that it is never safe to neglect the curvature as has been customary in the past. The middle-third theorem, therefore, does not hold for arched dams.

Introduction

The customary investigation of gravity dams is based on the assumption that the horizontal sections, such as $F\ G$ in Fig. 1, remain plane and that the modulus of elasticity remains constant, so that the stress, s_b, produced by a bending moment, M, is,

$$s_b = \frac{M}{I}\ y \qquad (1)$$

in which, I is the moment of inertia of the horizontal section, $F\ G$, about its center of gravity, and y is the distance from this center of gravity to the point in which the bending stress is s_b.

It is also customary to neglect the curvature of a gravity dam and assume that the horizontal section is a rectangle, as shown in Fig. 1, while actually it is trapezoidal, as shown in Fig. 2. It is the purpose of this paper to show that it is not safe to neglect the curvature and that this may lead to, and has led to, dams having tension in the up-stream face. Such a dam is not in fact a gravity dam, since it will not stand by gravity, but requires tension in the up-stream face.

Let B be the base width (Figs. 1 and 2) and H the height of a triangular section with a vertical up-stream face and let w be the weight of a cubic foot

Note.—Written discussion on this paper will be closed in December, 1927.

* Cons. Engr., Los Angeles, Calif.

of concrete, then if uplift is assumed as negligible and if the water level stands even with the crest of the dam, the section determined by,*

$$B = H\sqrt{\frac{62.5}{w}} \qquad (2)$$

will have zero stress at the heel when the reservoir is full, provided the dam is straight. If the dam is arched there will be tension in the up-stream face, on the assumption that the cantilever carries the entire load and, in general, this assumption will be the correct one and always on the safe side.

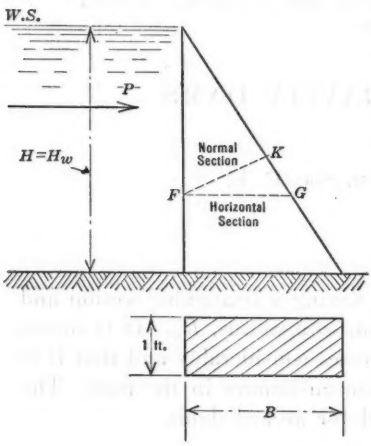


FIG. 1.

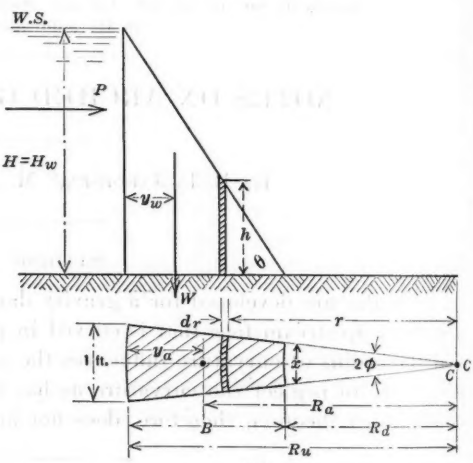


FIG. 2.

In deriving the ordinary bending formulas, it is assumed that normal sections remain plane when the shear deformations are neglected; but a horizontal section, such as $F\ G$ in Fig. 1, is not a normal section, as is $F\ K$. The writer has dealt elsewhere† with this question and the approximation introduced by substituting an oblique section for a normal section, but in order to avoid mathematical complications, the assumption that horizontal sections remain plane will be maintained. It is very essential, however, that the designing engineer bears in mind that the theory is not exact; then he will not be tempted to reduce the section to the theoretical minimum.

In this paper a triangular vertical section with a vertical up-stream face will be considered. It is customary to give the up-stream face a slight batter in order that there may be no tension in the down-stream face when the reservoir is empty. There can be no objection to such a batter. On the other hand, it is much more important to make sure that tension does not exist in the up-stream face when the reservoir is full, because if this results in a horizontal crack, the tension is thereby increased. Conversely, if there is tension in the down-stream face with reservoir empty, and a crack

* See, for example, the writer's discussion in *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 93.

† "Stresses in Multiple-Arch Dams," *Transactions*, Am. Soc. C. E., Vol. 87 (1924), p. 289; see, also, Professor Cain's discussion on p. 316. Also, Professor Résal's paper, "Formes et dimensions des grands barrages en maçonnerie," *Annales des Ponts et Chaussées*, II, 1919.

results, the tension is thereby reduced; the crack in the down-stream face is, therefore, self-limiting and is not likely to be more than a mere surface crack, while a crack produced by tension in the up-stream face is dangerous. Two fairly large dams have failed from tension in the up-stream face several years after they were built.*

Formulas for Stresses

Consider in Fig. 2 a volume 1 ft. wide at the up-stream face and limited by two vertical radial planes, instead of by two vertical parallel planes. Let R_u be the radius of the up-stream face; B, the base width of the section; R_d, the radius of the down-stream face; and $2\,\phi$, the central angle of the sector. Then,

$$2\,\phi = \frac{1}{R_u}$$

and the width of the down-stream face is,

$$x = 2\,\phi\,R_d = 2\,\phi\,(R_u - B) = 1 - \frac{B}{R_u} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The distance, R_a, from the center, C, to the center of gravity of the base is given by Merriman:†

$$R_a = \frac{2}{3}\,\frac{R_u^3 - R_d^3}{R_u^2 - R_d^2}\,\frac{\sin\phi}{\phi} = \frac{2}{3}\,\frac{\sin\phi}{\phi}\,\frac{R_u^2 + R_u R_d + R_d^2}{R_u + R_d}$$

$$= \frac{2}{3}\,\frac{\sin\phi}{\phi}\,\frac{(R_u + R_d)^2 - R_u R_d}{R_u + R_d} = \frac{2}{3}\,\frac{\sin\phi}{\phi}\left(2\,R_m - \frac{\left(R_m + \frac{B}{2}\right)\left(R_m - \frac{B}{2}\right)}{2\,R_m}\right)$$

$$= \left(R_m + \frac{B^2}{12\,R_m}\right)\frac{\sin\phi}{\phi}$$

in which, $R_m = R_u - \frac{B}{2}$, is the mean radius.

The distance from the up-stream face to the center of gravity of the base is then,

$$y_a = R_u - \left(R_m + \frac{B^2}{12\,R_m}\right)\frac{\sin\phi}{\phi} = \frac{B}{2} - \frac{B^2}{12\,R_m}\Big)\frac{\sin\phi}{\phi} + R_u\left(1 - \frac{\sin\phi}{\phi}\right)$$

The last term is always very small, because ϕ is small; for $R_u = 100$ ft., it becomes 0.0004 ft.; for $R_u = 500$ ft., it is 0.0000835 ft. Since y_a is approximately equal to $\frac{B}{2}$, the last term is negligible and $\frac{\sin\phi}{\phi}$ is always practically equal to 1, so that, very nearly,

$$y_a = \left(\frac{B}{2} - \frac{B^2}{12\,R_m}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

For a straight dam, $\frac{B^2}{12\,R_m}$ vanishes, so that $y_a = \frac{B}{2}$ as it should.

---

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Session 1907–08, Part II, p. 73. The two dams are the Habra Dam in Algiers and the Bouzey Dam in France.

† American Civil Engineers Handbook, Fourth Edition, 1920, Section 12, Article 23, p. 1471.

The volume of the dam sector limited by the two vertical radial planes (Fig. 2) is found by multiplying the area of the triangle, or $0.5\ B\,H$, by the length of the path described by the center of gravity of the triangle, as it revolves about the center, C. The length of this path is,

$$2\ \phi \left(R_u - \frac{B}{3}\right) = 1 - \frac{B}{3\ R_u} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

If w is the weight per cubic foot of concrete and all dimensions are in feet, the weight of the dam sector is,

$$W = \frac{w}{2}\ B\ H \left(1 - \frac{B}{3\ R_u}\right) \text{ lb} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

For a straight dam, $\frac{B}{R_u}$ vanishes and Equation (6) remains correct.

The position of the center of gravity of the volume is found by determining the moment of the volume about C and then dividing by W. The moment about C of the part of a cylindrical surface indicated in Fig. 2, is,

$$\text{Moment} = w\ h\ dr \times 2\ r\ \phi \times r\ \frac{\sin\ \phi}{\phi}$$

since the center of gravity of this thin cylindrical surface is given by $r\ \frac{\sin\ \phi}{\phi}$. The moment about C of the whole volume is, then,

$$M_c = \int_{R_u - B}^{R_u} 2\ r\ w\ \phi\ h\ dr\ r\ \frac{\sin\ \phi}{\phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

From Fig. 2,

$$\frac{h}{H} = \frac{r - R_d}{R_u - R_d}$$

or,

$$r = \frac{B}{H}\ h + R_d$$

Introducing this into Equation (7),

$$M_c = 2\ w \sin\ \phi \int_0^H \left(\frac{B}{H}\ h + R_d\right)^2 \frac{B}{H}\ h\ dh$$

$$= w\ H\ B \sin\ \phi \left(R_d^{\,2} + \frac{4}{3}\ B\ R_d + \frac{B^2}{2}\right)$$

The length of the path of the center of gravity of the triangle in Fig. 2, is, from Equation (5),

$$2\ \phi \left(R_u - \frac{B}{3}\right) = 2\ \phi \left(R_d + \frac{2}{3}\ B\right)$$

so that the weight of the volume of the dam is,

$$W = w\ \phi\ B\ H \left(R_d + \frac{2}{3}\ B\right)$$

and since,

$$\left(R_d + \frac{2}{3}\ B\right)^2 = R_d^{\,2} + \frac{4}{3}\ B\ R_d + \frac{4}{9}\ B^2$$

the distance from C to the center of gravity of the volume of dam, is,

$$r_w = \frac{M_c}{W} = \left(R_a + \frac{2}{3} B + \frac{B^2}{18 \left(R_a + \frac{2}{3} B \right)} \right) \frac{\sin \phi}{\phi} \quad \ldots\ldots (8)$$

Designate by y_w the distance from the up-stream face to the center of gravity of the volume, then, $y_w = R_u - r_w$, and since $\frac{\sin \phi}{\phi}$ is practically unity,

$$y_w = \frac{B}{3} - \frac{B^2}{18 R_u \left(1 - \frac{B}{3 R_u} \right)} \quad \ldots\ldots (9)$$

The moment of inertia of the trapezoidal base in Fig. 2 with reference to its own center of gravity is,*

$$I = \frac{B^3}{36} \left(\frac{x^2 + 4x + 1}{x + 1} \right) \quad \ldots\ldots (10)$$

in which, x is given by Equation (3), as,

$$x = 1 - \frac{B}{R_u}$$

By direct integration is found,

$$I = \frac{\phi}{4} (R_u^4 - R_d^4) - \frac{\phi}{2} (R_u^2 - R_d^2) R_a^2$$

The expression given in Equation (10) is much more convenient. If the water level is H_w ft. above the base, the horizontal water pressure is $31.25 H_w^2$ and the moment about the base is,

$$M_P = -\frac{62.5}{6} H_w^3 \quad \ldots\ldots (11)$$

The weight acting in the center of gravity of the volume may be considered as producing a vertical downward thrust equal to W and applied in the center of gravity of the base, and a moment,

$$M_w = W (y_a - y_w)$$

This moment acts in a direction opposite to the M_P produced by the water pressure; the resulting moment is, therefore,

$$M = -\frac{62.5}{6} H_w^3 + W (y_a - y_w) \quad \ldots\ldots (12)$$

From Equations (4) and (9),

$$y_a - y_w = \frac{B}{6} - \frac{B^2}{36 R_m \left(1 - \frac{B}{3 R_u} \right)} \quad \ldots\ldots (13)$$

The vertical stresses at the up-stream and down-stream faces, respectively, are then,

$$\left. \begin{aligned} s_u &= \frac{W}{A} + \frac{M}{I} y_a \\ s_d &= \frac{W}{A} - \frac{M}{I} (B - y_a) \end{aligned} \right\} \quad \ldots\ldots (14)$$

* See, Goodman "Mechanics Applied to Engineering" (1912), p. 86.

in which, A, the area of the base, is

$$A = B\left(1 - \frac{B}{2\,R_u}\right) = B\,\frac{1+x}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

In Equation (14) compression is positive and *vice versa*. The maximum compression, s_{max}, occurs at the down-stream face and is parallel to that face*:

in which,

$$\left.\begin{aligned} s_{max.} &= \frac{s_d}{\sin^2\theta} \\ \tan\theta &= \frac{H}{B} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Influence of Uplift

Assume that uplift is a maximum at the heel, decreasing linearly to zero at the toe. If the maximum uplift pressure is 62.5 k H lb. per sq. ft., the coefficient, k, measures the percentage, so that for $k = 1$† the maximum possible uplift is considered and for $k = 0$, there will be no uplift. The total uplift acting on the trapezoidal base is, by Equation (6),

$$U = \frac{62.5}{2}\,k\,B\,H\left(1 - \frac{B}{3\,R_u}\right) \text{ lb} \quad \ldots\ldots\ldots\ldots\ldots (17)$$

Its point of application is given by Equation (9), since the uplift may be considered as due to a triangular mass of water acting on the base. It follows that provision for uplift may be made by substituting, in Equations (12) and (13), $W - U$ for W, or simpler still, by reducing w accordingly. If, for example, $w = 150$ lb. per cu. ft. and full uplift is to be assumed, the new value is,

$$w' = 150 - 62.5 = 87.5 \text{ lb. per cu. ft.}$$

If w' is substituted in Equation (2) for w, this equation gives B direct. If, for example, $w = 150$, Equation (2) gives $B = 0.645\,H$; if $w' = 87.5$, that is, full triangular uplift, then $B = 0.845\,H$. Equation (2) is correct only for a straight triangular dam having water level with the crest.

The foregoing formulas are correct for any radius of curvature and, therefore, also for a straight dam for which the radius of curvature is infinitely great.

Example

When the radius of curvature has a finite value, the relation between the height and the required base width can not be reduced to a simple formula, such as Equation (2), which applies only to a straight dam. In order to ascertain the influence of curvature it is more convenient to consider an actual case—to compute the stresses on the assumption that the dam is straight and then to re-compute them for an arched dam having the same

* "Stresses in Masonry Dams," by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 208.

† $k = 1$ is a limiting condition, which may be approached but cannot be reached, since the weight of the concrete exceeds the uplift and, therefore, some part of the base must have contact with the foundation.

section. Consider, therefore, a horizontal section of the Exchequer Dam of the Merced Irrigation District, Merced, Calif.* At Elevation 437.5, the dimensions are, $H = H_w = 272.5$ ft.; $B = 171.4$ ft.; $R_u = 674.7$ ft.; and $R_m = 589$ ft. The actual section is not triangular with a vertical up-stream face, but has a definite top width, and the up-stream face is battered, so that at the elevation considered it lies 1.97 ft. up stream from the crest. The dimension, B, used in the calculations is the actual total base width; it has merely been shifted 1.97 ft. down stream. The approximation involved in substituting the ideal triangular section for the actual section is a very close one.

If the dam were straight the value of w, giving zero stress at the heel with the reservoir full, may be obtained from Equation (2), since the dimensions are known. This gives $w = 160$ lb. per cu. ft.—an unusually high value for designing a gravity dam; but if w is assumed as less, a straight dam of the section given would show tension in the up-stream face. Assuming, now, $w = 160$, $y_a = 81.54$ ft.; $W = 3\,418\,000$ lb.; $y_w = 54.49$ ft.; $I = 364\,000$ ft.^{4}; $x = 0.746$ ft.; and from Equation (12),

$$M = -210\,800\,000 + 578\,000\,w = -118\,400\,000 \text{ ft-lb.}$$

The area of the base is $A = 149.5$ sq. ft., and the vertical stresses at the heel and toe, respectively, are, $s_u = -25.9$ lb. per sq. in. (tension) and $s_d = 362.0$ lb. per sq. in., and the maximum stress is $\frac{362}{0.847^2} = 504$ lb. per sq. in. If the dam had been straight, the stresses would have been $s_u = 0$, and $s_d = 303$ lb. per sq. in. If $w = 152.0$ is assumed, the actual section ($R_u = 674.7$ ft.) gives $s_u = -40.5$ lb. per sq. in (tension).

Consider that there is 50% uplift at the heel and none at the toe, and a linear decrease; also, assume $w = 152$, so that,

$$w' = 152 - 31.25 = 120.75 \text{ lb. per cu. ft.}$$

Then, for $H = H_w = 272.5$ and a straight dam, from Equation (2), $B = 196$ ft. If, as before, $R_u = 674.7$ ft., $H = 272.5$ ft., but $B = 196$ ft., so that $R_m = 576.7$ ft., then the values will be $x = 0.7095$ ft.; $W = 2\,912\,400$ lb. (this is actually the weight less the uplift, since w' and not w was used); $M = -121\,607\,000$, since $y_a - y_w = 30.618$ ft.; the area, $A = 167.53$ sq. ft.; $y_a = 92.45$ ft.; $I = 531\,160$; and the stress in the up-stream face with reservoir full is, $s_u = -26.3$ lb. per sq. in. (tension).

General Remarks

From this example evidently it is never safe to neglect the curvature; the higher the dam and the smaller the radius, the larger is the error committed when the dam is computed as straight.

It is also evident that an arched gravity dam will receive some support from the arch and, in most cases, probably sufficient support to prevent actual failure. As gravity dams are usually constructed, however, the arch cannot act until the cantilever has been broken, and in that case uplift would

* A section of this dam is given in *Western Construction News*, January 25, 1926, p. 25.

be present. The writer has discussed this phase in his paper entitled "Stresses in Thick Arches of Dams",* and also made mention of the shrinkage cracks in the Mulholland Dam, which proves that the dam is not acting as an arch dam. In a properly designed gravity dam the shrinkage cracks are of little consequence; but when tension is present in the up-stream face the dam should be computed as an arch dam, since it does not appear safe to rely on tension in the concrete, especially not across construction joints.

In the writer's opinion the most important condition to impose on a gravity design is that there must be no tension in the up-stream face when the water surface is at the maximum height; it is also well to bear in mind that the assumption of no uplift is the least conservative that can be made, and while no doubt occasionally it is safe, especially when proper drainage is provided, there is always the possibility, not to say certainty, that the drainage will fail at least partly. For reservoir full, the stress at the toe is of importance only for very high dams, but the stress at the heel is by far the most important.

The writer is fully aware of the uncertainties involved in determining the stresses near the foundation of a gravity dam. This uncertainty arises partly from the unknown distribution of the shear near the base.† This phase of the subject has been purposely excluded, because it was desired to show the influence of curvature when the ordinary assumptions were maintained. As Professor Résal points out, a straight gravity dam may also act as an arch dam, since, in general, secondary horizontal arches may be found for any straight gravity dam. The extent of arch action will depend on the width of the site and on the method of construction; whether it was pressure grouted after the concrete had cured and the chemical heat had been dissipated; whether the dam was closed as late as possible; or, finally, whether no attempt at all was made to secure arch action in case the uplift for any reason should prove to be in excess of the assumption.

* *Proceedings*, Am. Soc. C. E., February, 1926, Papers and Discussions, p. 206.

† Professor Cain calls attention to this uncertainty in the opening sentence of his paper previously referred to and Professor Résal in the paper referred to deals at some length with this subject.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

PREPARING THE GROUNDWORK FOR A CITY: THE REGRADING OF SEATTLE, WASHINGTON*

By Arthur H. Dimock,† M. Am. Soc. C. E.

Historical

In the fall of 1851 a little band of pioneers landed on a point of land between Elliott Bay and Puget Sound. With the optimism of the adventurer they promptly christened their new settlement "New York". They spent the winter crowded into one small log cabin and having for near neighbors nearly a thousand Indians. Their time was chiefly occupied in exploration. Before determining their future course, and not being in possession of the charts made by Captain Wilkes in 1841, they made a hydrographic survey of Elliott Bay, their instruments consisting of a clothes-line and two horse-shoes. When the shoes failed to reach the bottom, they knew they had ample depth for ships. In the spring of 1852, the majority of these pioneers removed to the east side of Elliott Bay. One of the reasons impelling their removal was that the steeper slope of the shores on the east side of the bay made possible much shorter wharves than the long flat slopes of "New York". One of their number, Carson D. Boren, built the first cabin in Seattle near the intersection of Second Avenue and Cherry Street, where the Hoge Building now stands. By 1853 they had constructed a saw-mill near the foot of Yesler Way and laid out and platted the new town site.

There were other pioneers on Puget Sound in those days and these used to wag sapient heads and remark "Alki New York", "Alki" being Chinook for "by and by", or "a long time ahead". Finally, "New York" was dropped, and to this day the place retains the name of Alki. For several years, however, "New York" and Seattle were rival towns.

Note.—Written discussion on this paper will be closed in December, 1927.
* Presented at the meeting of the City Planning Division, Seattle, Wash., July 15, 1926.
† Cons. Engr., Seattle, Wash.

In 1855 Governor Isaac I. Stevens, who had charge of the explorations for a Northern Pacific railroad which were being made by Captain George B. McClellan, and others, for the United States War Department, of which Jefferson Davis was then Secretary, recommended the selection of Seattle as the Western terminus because of its "unequalled harbor". This report gave a great impetus to the new settlement and raised the expectations of the pioneers to the highest pitch.

Topography

The general topography of the new city and vicinity is shown on Fig. 1. Taking First Avenue and Yesler Way as the center in those early days, there lay to the south about 1 500 acres of tide flats, bounded on the east and west by steep slopes and intersected by the Duwamish River. (See Fig. 1.) The flat low-lying valley of this river, about 1 mile in width, extends southeasterly for about 8 miles, where it opens into far wider valley lands reaching to Tacoma, Wash. North of the central point are steep slopes coming directly down to the water-front, flattening in the vicinity of Pike Street to form a considerable level area about 100 ft. above high tide. Immediately north of Pike Street, and a little to the westward, lay Denny Hill, with a maximum elevation of about 240. To the east of Denny Hill was a valley extending to Lake Union, also bounded on the east by steep slopes.

About 1 mile from Pike Street and less than 2 miles from Yesler Way is the south end of Lake Union, a body of water of about 800 acres, 1 mile in length due north and south, and ½ mile in width, lying 8 ft. above high tide. It is bounded on the east and west by steep hills, but has level land at its southern end. It possessed an outlet to Puget Sound through Salmon Bay and was separated from Lake Washington by a narrow ridge. From the earliest times the pioneers dreamed of connecting these lakes with the Sound by a ship canal, which has since been accomplished. Between Elliott Bay and Lake Washington, about 20 miles long and 2 to 3 miles wide, was a series of hills and valleys. North of Lake Union was an extensive, rolling plateau.

Such, in brief, was the general character of the site selected by these pioneers for the city of their dreams. It possessed many elements corresponding to their vision. First, was the sheltered harbor with ample depth of water, which had caught the eye of McClellan, and the bays and lakes, which, when connected, were to form an internal system of water communication. Then, there were the tide lands which, when filled, were to form a manufacturing district with easy access to rail and water. The level lands in the neighborhood of Pike and Pine Streets, when extended by proper grading operations, were to become the retail business section and between the manufacturing and retail areas, on the slopes of the hillside, was ample room for the buildings of the office and financial districts. On the hills, surrounded by sea and lake and encircling snow-clad mountains, were home sites of rare beauty. The instincts of the pioneer led them aright. Although not without its difficulties, the situation of Seattle is unique among the cities of the world.

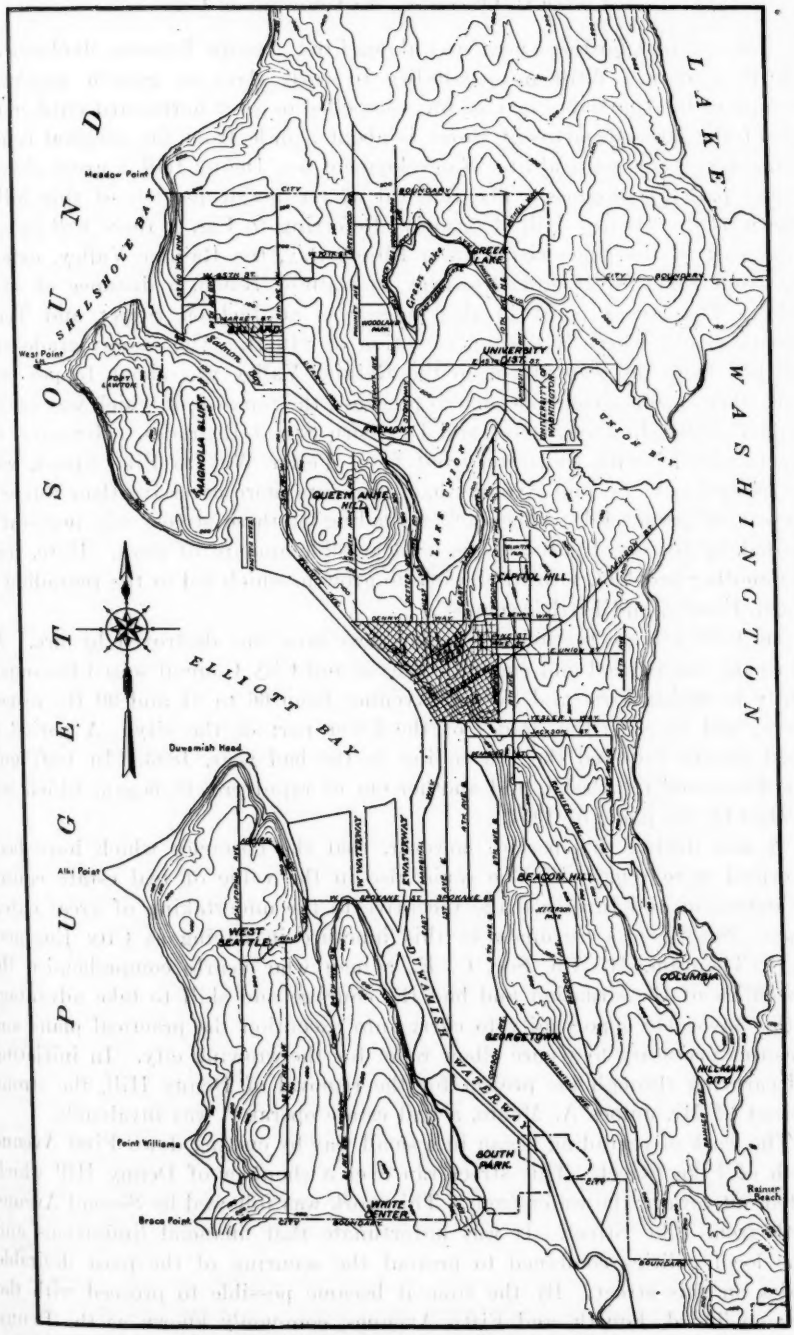


FIG. 1.—TOPOGRAPHIC MAP OF SEATTLE, WASH.

Physical Obstacles to Growth of City

The original business area was along First Avenue between Jackson and Cherry Streets. With no possibility in early days of growth southward because of the tide flats, the city was compelled to grow northward until to-day one of the large department stores is about a mile from the original center. Lying across this natural line of development was Denny Hill, a great obstacle in the path of progress. The removal of the major portion of this hill is known as the "Denny Hill Regrade". Referring to Fig. 1, there will be seen lying east of the ridge rising from Elliott Bay, the Rainier Valley, extending from Pike Street and Broadway to Rainier Beach, a distance of seven miles. The lowest point in this ridge was at Jackson Street and Tenth Avenue South where it had an elevation of 210 ft. The easiest grade over this hill from the water-front to the Rainier Valley was about 15 per cent. Here, then, was a second obstacle to progress, the removal of which was accomplished by the Jackson Street and Dearborn Street Regrades. Moreover, the streets parallel with the water-front from Yesler Way to Pike Street, with the exception of Second Avenue, had gradients more excessive than was convenient or permissible. The earlier grading of these streets was necessarily limited, by reason of costs, to the least possible amount of work. Here, then, was another problem, a third obstacle to growth, which led to the regrading of Third, Fourth, and Fifth Avenues.

In 1889 a large section of the business area was destroyed by fire. An unusually far-sighted and energetic Mayor and City Council seized the opportunity to widen First and Second Avenues from 66 to 84 and 90 ft., respectively, and to raise the grades of the lower part of the city. A period of rapid growth followed the fire ending in the bad year, 1893. In 1897 gold was discovered in Alaska, and another era of rapid growth began, which was checked by the panic of 1907.

It was during this period, however, that the obstacles which have been described were removed. The rapid rise in the value of real estate caused the optimism and furnished the incentive to the undertaking of great enterprises. Seattle was fortunate in this juncture in having as City Engineer R. H. Thomson, M. Am. Soc. C. E., a man who clearly comprehended the necessities of the situation and had the courage and skill to take advantage of this favorable opportunity to carry into execution the practical plans and measures necessary to secure elbow room for the growing city. In initiating and carrying through the project for the removal of Denny Hill, the strong support of Mr. James A. Moore, a real estate operator, was invaluable.

The work of regrading began in a small way by cutting down First Avenue north of Pike Street. This street ran over a shoulder of Denny Hill which jutted out toward the water-front. This work was followed by Second Avenue north from Pike Street. It was unfortunate that financial limitations and some local politics combined to prevent the securing of the most desirable grades on this street. By the time it became possible to proceed with the work on Third, Fourth, and Fifth Avenues, commonly known as the Denny Hill Regrade, a considerable number of new buildings had been erected along

Second Avenue. This made it practically necessary to conform the grades of the new work to that of Second Avenue and led to the creation of a hump at Second Avenue and Virginia Street, which subsequent efforts have failed to remove.

Street Grades.—Under modern conditions, however, the necessity for easy gradients has diminished. These gradients were determined formerly by considering the load a team of horses could pull, and those pavements were selected which afforded the best footing. The motor vehicle has made old theories seem very ancient indeed. It is still true that easy grades are better than steep ones, but the limits have been greatly widened. There is one street in Seattle which was paved with asphalt on a maximum grade of 12%, on the theory that it would never be used anyway; but it has become the main access to the First Hill District. It requires a sprinkling of sand under certain weather conditions, but the capitalized cost of this expense would go a very small way toward paying the cost of securing a better gradient. The ease and comfort of the lowly pedestrian is a more potent factor in limiting the growth of a business district and in determining limiting gradients than motor traffic. Traffic now will avoid detours and seek the steep short-cut wherever possible. In order to obtain a 5% gradient, on one of the parkways, a series of hair-pin turns, reminiscent of stage coach days on mountain roads, was used. To-day, these turns are a source of inconvenience and even danger. A more direct route with a 12% gradient would be preferable.

Some years ago the writer recommended the use of escalators on the steep cross streets of the business district of Seattle. The practical effect of these escalators would be equivalent to the flattening out of these grades to pedestrians. Some day, owners of the property on these steep streets will awaken to the value of this mechanical means of overcoming the disadvantages of their situation. When the first street is properly equipped with this means of easy access the remainder will hasten to follow the lead.

The rates of grade on the different regrade projects are shown on Figs. 2, 3, and 4. The maximum grade in the Denny Hill District is 5% on the north and south streets and 6.7% on the east and west streets. In the Jackson Street District the grade on Jackson Street itself is 5%, with somewhat steeper grades on the less important streets. Dearborn Street possesses a 3% maximum, but no advantage seems to accrue to it because of this better rate. On the streets in the office district the maximum rates are as follows: First Avenue, 7%; Second Avenue, 5%; Third Avenue 3.7%; Fourth Avenue, 4.7%; and Fifth Avenue, 6 per cent.

The maximum depth of cuts on the Denny Hill project was 110 ft., on the Jackson Street project, 84 ft., and on Dearborn Street, 108 ft. The quantities of earth removed were, respectively, 5 400 000 cu. yd., 3 400 000 cu. yd., and 1 600 000 cu. yd.

Material Encountered.—Fortunately, these hills were composed of sand, gravel, clay, and hardpan. They were the product of glaciers which, in the Pleistocene Age, covered the site of Seattle to a depth of several thousand feet. Geologists state that there were two such ice sheets, each originating in the

Cordilleran center and coming down from the north. They were separated in time by a considerable inter-glacial period. Little of the deposits of the first sheet are exposed, but the handiwork of the second constitutes the foreground of much of the scenery of Puget Sound. The glacier extended to a point about 70 miles south of Seattle. It dammed the natural outlet of the Sound through the Straits of Juan de Fuca. When it began to recede and melt, the water, denied its normal outlet through the Straits, formed a lake or lakes into which flowed many of the rivers of the Sound Basin. The outlet for this water was over a low divide to the south and down the Chehalis River to the Pacific Ocean.

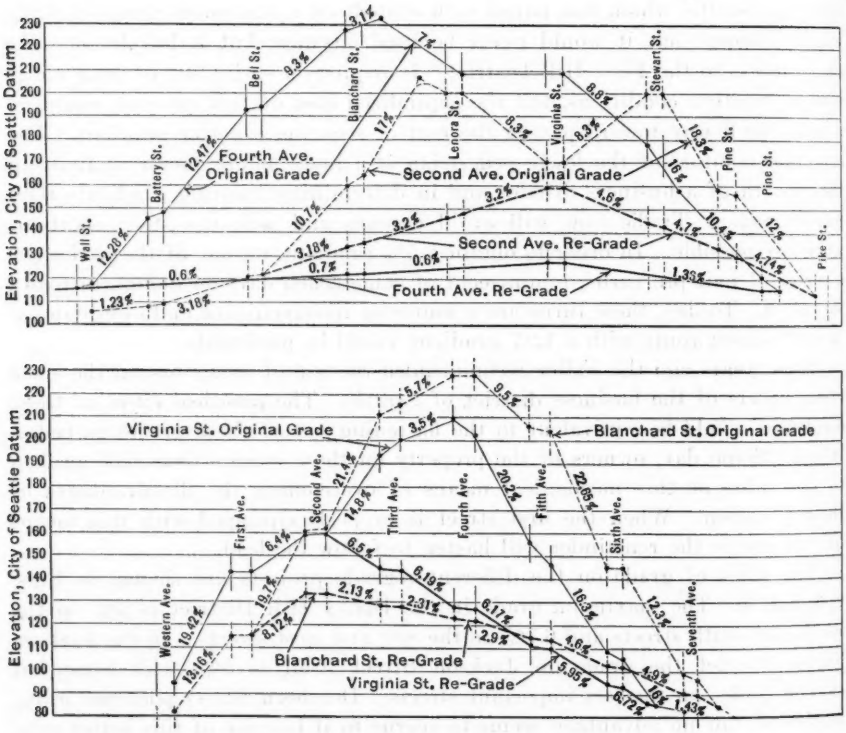


FIG. 2.—PROFILES OF DENNY HILL REGRADE.

The outwash from the glacier formed extensive gravel plains south of Tacoma which now yield a large part of the sand and gravel required for building and engineering construction in Seattle, Tacoma, and other Puget Sound cities. As only the very best and hardest of the rocks survived the action of the glacial mill, the quality of this material is exceptionally high. The retreating glacier also deposited a layer of material named Vashon Till by geologists and commonly known as hardpan. Its thickness varies from 2 or 3 to 20 ft., or more. Beneath are sand and gravel and extensive deposits of a hard blue clay, laid down in the glacial lakes from materials accumulated by

the first glacier. It constitutes the foundation of much of the construction work. Rock occurs only in a very few places in the city and not at all in the business district. This, then, was the material to be moved, and as the cost of handling the large volume of excavation by orthodox methods would have been prohibitive, a new and cheaper method had to be discovered.

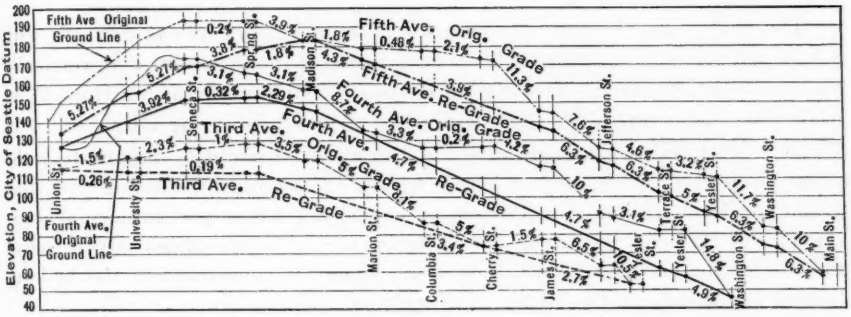


FIG. 3.—PROFILES OF THIRD, FOURTH AND FIFTH AVENUE REGRADES.

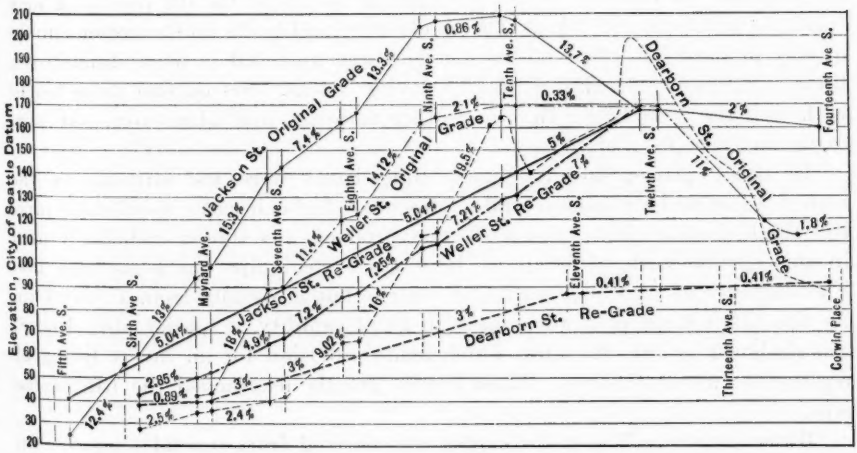


FIG. 4.—PROFILES OF JACKSON AND DEARBORN STREET REGRADES.

Application of Hydraulicking.—Mr. William H. Lewis, a lawyer more interested in engineering and constructive work than in law, possessed property interests in the Rainier Valley. This property, although close in, was separated from the business section by the ridge on Jackson Street previously described. The removal of this ridge at a cost within the ability of the property owners to pay became to him a topic of absorbing interest. His attention was called by W. C. Morse, M. Am. Soc. C. E., and by Mr. James Hopkirk, to the successful use of the method of hydraulic sluicing in extensive gold-mining operations at various points on the Pacific Coast. The method, however, had never been applied to the problem of moving large volumes of clay

and hardpan on a city grading job. Many said it could not be done. Bankers refused their support, but Mr. Lewis, by his energy and enthusiasm, overcame all difficulties, worked out the problem, secured the approval of the property owners and of the City Council, and finally organized a company which obtained the contract and successfully completed the Jackson Street and Dearborn Street projects.

The older form of giant was adapted to the new work by equipping it with a ball-and-socket joint to enable it to swing both horizontally and vertically as conditions required. Detachable nozzles varying in diameter from 2½ to 3½ in. were also provided. The water had a pressure at the nozzle of 75 to 100 lb. With this equipment much of the material could be removed by the water alone; but the harder materials required loosening by blasting with low-grade powders. It was soon learned by experience where preliminary shattering of the material increased the efficiency of the water sufficiently to offset the cost of the blasting. Figs. 5, 6, and 7 show the magnitude of the hydraulicking operations on the Denny Hill regrade.

The material when broken by jet and powder was washed down to sluice-boxes provided with fan-shaped approaches and openings screened by bars or "grizzlies" to keep out roots and material too large for the pipes. A man stationed at the "grizzly" kept it from clogging and broke up the larger chunks of clay or hardpan. Part of the material was conveyed in open sluice-boxes, lined with wooden blocks, similar to paving blocks. Set on end these blocks withstood the wear better than any other material, and when worn out were easily and cheaply replaced.

In many places, however, open sluice-boxes were not suitable as the material had to be conveyed under pressure. In such cases wood-stave pipe, like the hydraulic giants, also a Western product, was almost exclusively used in this earlier work, although of later years steel pipe has come into use. The diameter of such pipe varies, of course, but is usually from 18 to 24 in. As this plain wood pipe wore out with great rapidity, a special pipe, Fig. 8, was designed by Mr. Hopkirk, which made provision for an invert lining of replaceable wooden blocks. These blocks greatly lengthened the life of the pipe.

Water Supply.—The water supply was derived from several sources. On the Jackson Street and Dearborn Street work surplus water from the Cedar River water supply system of the city was turned into an abandoned reservoir with a capacity of about 4 000 000 gal., situated about 1 mile from the work. From the reservoir this water was fed to the giants through steel and wood stave pipe. From 6 000 000 to 10 000 000 gal. per day were thus supplied at a charge of $15 per 1 000 000 gal. This, however, was not sufficient. A pumping plant was constructed on Elliott Bay having a capacity of 12 000 000 gal. per day, consisting of four 10-in., 5-stage pumps direct-connected to two 650-h.p. motors. As the tide has a range of 16 ft., the pumps were set somewhat below extreme high tide, and provision was made for a water-tight floor and wall. The discharge was made through 24-in. wood stave pipe having staves 1⅝ in. thick and bound with No. 1 wire spaced ⅞ in.

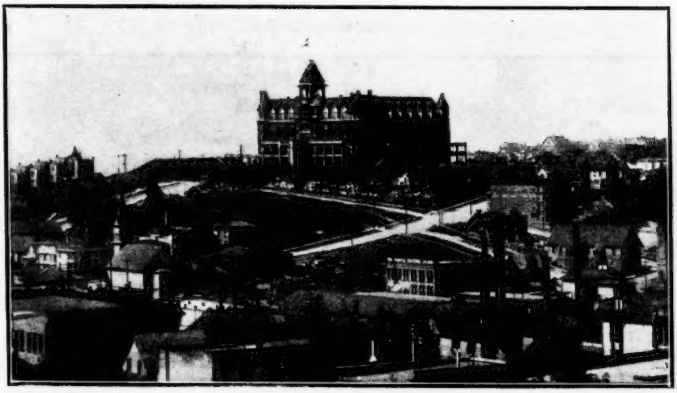


FIG. 5.—VIEW OF OLD DENNY HOTEL, SEATTLE, WASH., AND VICINITY.

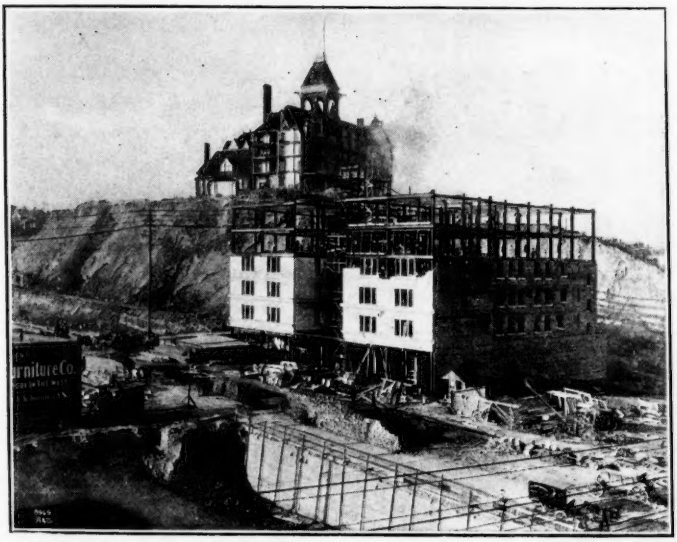


FIG. 6.—DENNY HOTEL PARTLY DEMOLISHED. NEW BUILDING ON COMPLETED PART OF REGRADE.



FIG. 7.—VIEW OF REGRADING OPERATIONS ON DENNY HILL.







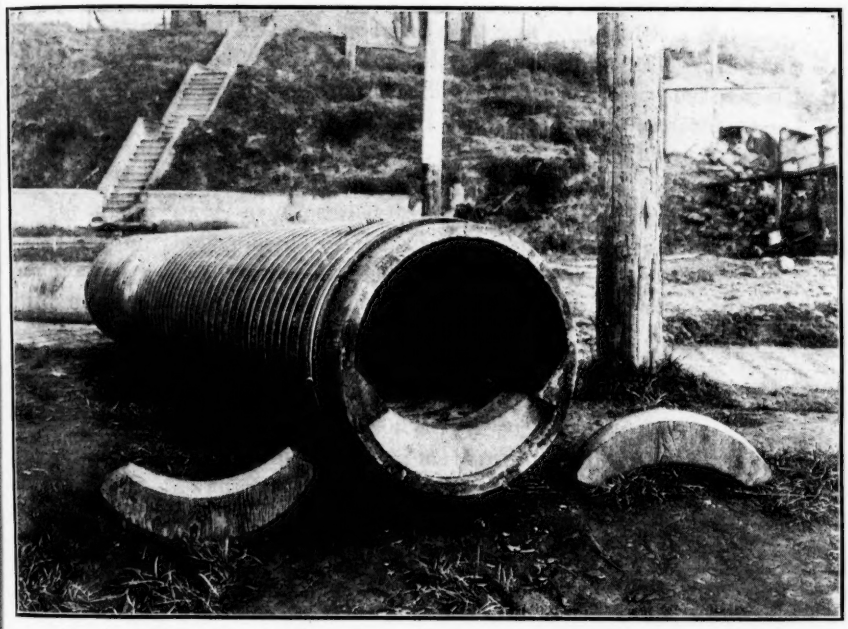


FIG. 8.—SECTION OF NEW SPOIL PIPE, REGRADING OPERATIONS, SEATTLE, WASH.

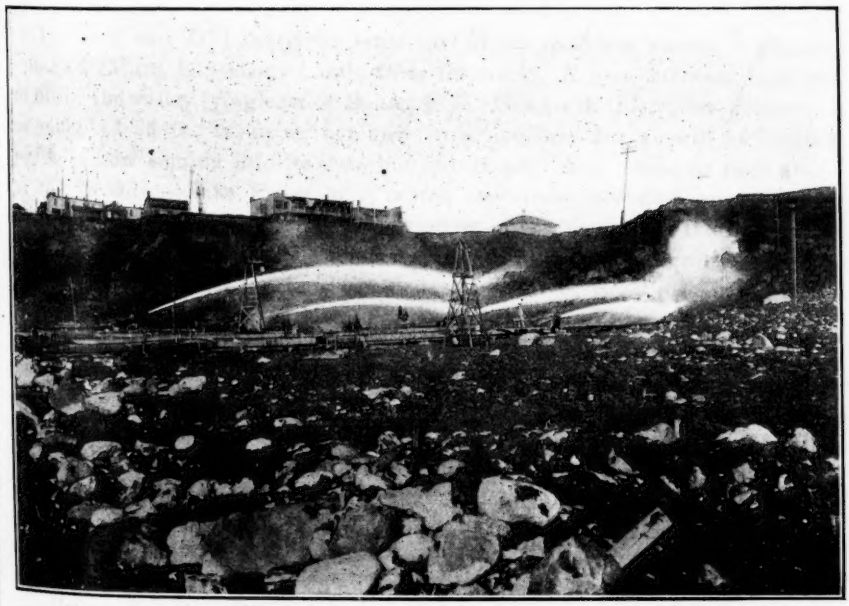


FIG. 9.—JACKSON STREET REGRADE, SEATTLE, WASH., SHOWING FOUR GIANTS OPERATING IN CLAY OVERLAID BY HARDPAN OR GLACIAL TILL.



On the Denny Hill work, city water was not available. The supply was from two sources. On the water-front a 4-stage pump with a 850-h.p. motor delivered 5 000 000 gal. per day. A second plant of three units was constructed on Lake Union about 1 mile from the work. Two units consisted of two 10-in., 4-stage turbines each, with a 650-h.p. motor. The other unit was a 15-in., 4-stage pump with a 650-h.p. motor. The total capacity was 12 000 000 gal. per day. Here, also, wood stave pipe conveyed the water to the giants.

The quantity of spoil carried by the water varied within wide limits, depending on the gradients of the discharge pipes and the character of the material. On Jackson Street the average was 6.75% of the volume of water. With a water supply of 20 000 000 gal. per day, this would mean 10 000 cu. yd. of material moved. A large allowance, however, on such work must be made for changing pipe lines, moving giants, clogging of discharge pipes, and all the innumerable petty delays incident to such work. On Jackson Street the quantity moved averaged about 4 000 cu. yd. per calendar day.

Disposal of Spoil.—In the Jackson Street regrade all the material was used to fill in the adjacent tide lands. The grades of some of this area were raised about 40 ft. In this, as in the Denny Hill project, provision was made in the specifiations for the removal of earth from private property at the same price as that paid by the City, the contractor being required to accept a lien on the property payable in ten annual installments, as in the case of the assessment for the street work. This provision applied also to the fills. The result was a substantially unanimous acceptance of this provision by the property owners, there being, however, a few exceptions.

In the Denny Hill District a large part of the spoil was wasted in the deep water of Elliott Bay about ½ mile from the work. A part, however, was used to fill in the valley lying east of Denny Hill. Grades in this valley were raised as much as 25 ft., resulting not only in a fine area for growth of business which is now coming into its own, but also in providing access by easy grades to the Broadway-Pike Street area, which has since developed into the principal motor-vehicle district. This by-product of the removal of the Denny Hill is proving to be even more valuable as affording scope for growth than the original project itself, as the largest development of Seattle is in a general northeastward direction and this valley lies almost directly along the axis of this movement. Another reason is that the Denny Hill District has been hampered in attaining the most complete benefit from its heroic work because the streets were originally laid out parallel with the salt-water front. This was perfectly natural for the pioneers to do, as they could not, at that time, visualize a removal of this hill, and they failed to grasp the northeastward trend which business later would take. Consequently, they extended this platting as determined by the direction of the salt-water shore-line up and over Denny Hill and even across a part of the valley. The result was that these streets diverge about 45° from their proper direction. A few streets paralleling the shore line and the remainder paralleling the section lines would have been almost ideal in its conformity to present necessities, for the

valley lies almost due north and south. This failure to perceive the future development, therefore, has given rise to several perplexing and difficult street rectifiation problems which are now vexing the City Planning Commission and the City Council.

A part of the excavation was carried on by steam shovel and dump cars. Much material was conveyed in this way to sluice-boxes, whence it was washed down to the Bay. Other parts were used to fill in the valley streets as outlined. In the Jackson Street District the low ground was brought up to basement level by hydraulicking (Fig. 9), and the street fills brought up to grade by the shear-board method, shown on Fig. 10, the slopes being on the private property. This method, which was specially devised to meet the conditions of this work, consisted of driving stakes against which were placed a couple of 1-in. boards. These stakes were braced to the step below. The steps were, say, 1½ ft. high and 2 ft. wide. The earth was then sluiced in nearly to the top of the boards when another step was made, and so on to the top. The steps varied somewhat in height and width, depending on conditions. Much of the lumber used in these steps could be used over again, and it proved to be a cheap and efficient method of building up the fills. In a few cases wooden pile bulkheads were constructed.

On Third, Fourth, and Fifth Avenues, between Yesler Way and Pike Street, in the central business district, all excavation was carried on by steam shovel and cars. This area, because of the character of the adjoining buildings, did not permit the use of the sluicing method. Crossing this district were two cable street-car lines, affording access to important residence districts. When the excavation of adjoining areas had proceeded sufficiently, these cable roads were blocked up on timber cribwork. After the excavations had been completed, they were lowered by degrees during the night. In this way traffic was maintained almost continuously. Electric street-car traffic on the north and south streets was shifted from one street to another and handled by various other expedients. The carrying on of this work in these streets greatly inconvenienced the stores and business houses. Although every possible aid was extended to the owners by the City authorities, nevertheless, there must have been considerable loss in many cases. This was a part of the price paid for the reshaping of these streets. In the Denny Hill and Jackson Street Districts, however, the cuts were so great that the buildings were wrecked, thus leaving the area entirely free for the sluicing operations. The most conspicuous example of this razing of buildings was the Denny Hotel, a brick structure of six or seven stories.

The total quantity of earth removed on this regrading work was approximately 14 000 000 cu. yd. A part of Denny Hill was not included in the original work, but provision is now being made for the full completion of this project. A great deal of such work has been done since by the hydraulic method, but it has not possessed the magnitude and spectacular features in depth of cut and volume of material moved, of the Denny Hill, Jackson Street, and Dearborn Street projects.



FIG. 10.—VIEW SHOWING SHEAR-BOARD METHOD OF CONSTRUCTING EMBANKMENTS.



FIG. 11.—VIEW SHOWING COMPACTED GRAVEL USED IN PORTLAND, ORE., FOR CONSTRUCTION OF HIGH EMBANKMENTS BY HYDRAULIC PROCESS.

Building Embankments by Hydraulic Process

An interesting development of the art of hydraulic sluicing is the construction of embankments which are entirely free from subsequent settlement. Fills have been made in Portland, Ore., 40 to 50 ft. deep, on which houses were built shortly after the fills were finished. These houses showed no more effects from settlement than those built on original ground. The University of Washington Stadium was built by the same method. The construction of the concrete work followed immediately on the completion of the fill, and no cracking due to settlement has occurred.

For the successful use of this method, suitable material must be available. Earth containing a large percentage of clay cannot successfully be used. The process reverses the theory of hydraulic construction of dams which retains the fines in a central pool, and allows these fines to run off. The water must be kept moving at the correct velocity to prevent deposition of the fines while it drops the coarser material. Fig. 11 is a view of compacted gravel banks, used in Portland, Ore., for the construction of high embankments. Such embankments drain very rapidly and form extremely compact and solid fills. The process is somewhat similar to the construction of the slopes of the hydraulic-fill dam.

It will be noted that the dreams of the original pioneer settlers for a great city, which they visualized, perhaps, somewhat hazily and indistinctly, required for their initial realization the development of pioneer engineering methods. Hydraulic sluicing, the hydraulic giant, wood-stave water pipe, wood-stave pipe lined with blocks for carrying spoil, and the shear-board method of building up embankments, are all the product of Western conditions.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

RE-ARRANGEMENT OF A BUSINESS DISTRICT: CHANGES IN RECENT YEARS IN PITTSBURGH, PENNSYLVANIA*

By Nathan Schein,† M. Am. Soc. C. E.

Incorporated as a city since 1816, and situated at the confluence of the Allegheny and Monongahela Rivers, uniting to form the Ohio, Pittsburgh, Pa., is one of the greatest industrial, financial, and distributing centers. The city proper, which has an area of 46.93 sq. miles, is divided by the rivers into three principal sections. The largest and most populous section known as "Peninsular Pittsburgh", or the "Old" City, occupies the territory between the Monongahela and Allgheny Rivers, and includes the down-town or business district. The territory south of the Ohio and Monongahela Rivers is known as the South Side, and that north of the Ohio and Allegheny Rivers, formerly the City of Allegheny, is known as the North Side.

The city ranks eighth in population, but the area included within a 10-mile radius and including that known as the Pittsburgh District, ranks fifth among the metropolitan districts of the United States. The down-town, or the business, section is substantially that part of the city known as the Point District, and is bounded by the two rivers and by Grant Street on the east.

The street plan in this district consists of a low-level section in which three main streets, Duquesne Way, Penn Avenue, and Liberty Avenue, are laid out parallel to the Allegheny River, with intersecting streets at right angles. To the east of Liberty Avenue, the streets and avenues are parallel to the Monongahela River, with Market, Wood, Smithfield, and Grant Streets intersecting at right angles.

The basic problem in this section is that of means of transportation, especially the problem of the street plan. Prior to 1904, this district consisted

Note.—Written discussion on this paper will be closed in **December, 1927.**

* Presented at the meeting of the City Planning Division, Seattle, Wash., July 15, 1926.

† Div. Engr., Dept. of Public Works, Pittsburgh, Pa.

of narrow streets and ways or alleys. There was no street or avenue wider than 60 ft. except Liberty Avenue of 80 ft. (referred to subsequently), and most of them were 40 ft. wide; all the alleys or ways were 20 ft. wide.

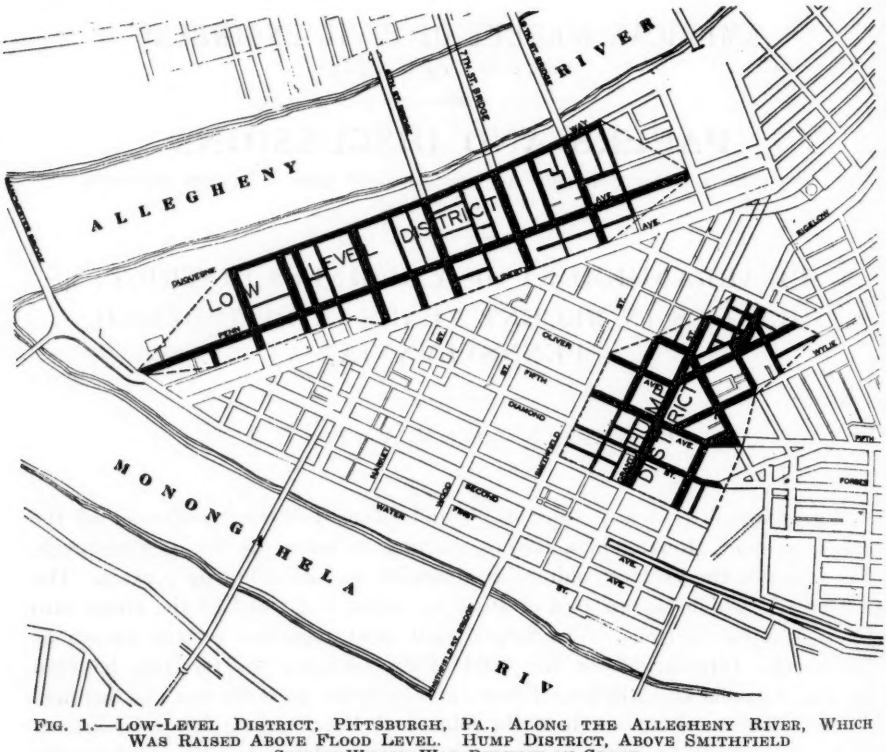


FIG. 1.—LOW-LEVEL DISTRICT, PITTSBURGH, PA., ALONG THE ALLEGHENY RIVER, WHICH WAS RAISED ABOVE FLOOD LEVEL. HUMP DISTRICT, ABOVE SMITHFIELD STREET WHICH WAS REDUCED IN GRADE.

In 1904, the City realized the need of more street area in the down-town section and widened Diamond Street to 50 ft. from Smithfield Street to East Diamond Street, and, at the same time, Oliver Avenue was widened to 44 ft. from Smithfield Street to Liberty Avenue. In 1905, the Pennsylvania Railroad Company removed its tracks from Liberty Avenue which then became the first street of this city with a free roadway of 48 ft. between curbs. These were the only improvements that had been made in the re-arrangement of the streets in this district and the problem of transportation began to be seriously felt.

The various civic and trade organizations as well as the Department of Public Works began to study this problem. In 1910, Mr. Frederick Law Olmstead, in a report entitled "Pittsburgh, Main Thoroughfares and Down-Town District", made recommendations for the general improvement of the city, and called attention to the importance of proper and adequate city planning. This study resulted in the creation of the City Planning Department in 1911, which gives the City the power to bring about proper planning and provided a body for its administration.

In 1911, a broad and comprehensive program of street improvements was started. These projects included the improvement of main thoroughfares, particularly those leading from the business district to outlying sections; opening of new highways; elimination of grade crossings; raising of low-level areas; reduction of grades; and widening of business thoroughfares. The two methods of city planning carried out were the reduction of grade in the "Hump" District and the raising of the low-level districts above flood stage. The location of these districts is shown in Fig. 1.

Reduction of Grade in the Hump District

The improvement of streets in what is known as the "Hump" District, completed in 1913, involved the lowering of grades in an area of about 40 acres in the down-town business district, the opening of a new thoroughfare, and the widening of a number of streets. The necessity for this work arose from the fact that the grades of Fifth, Oliver, and Sixth Avenues, over which the greater part of the east-going traffic passes, were very steep. The traffic congestion on Fifth and Sixth Avenues, and the necessity for providing an outlet for Grant (now Bigelow) Boulevard, demanded the opening of an additional parallel route which was accomplished by widening Oliver Avenue. Other street widenings for the accommodation of through traffic and to meet local requirements were also made.

Twice before the grade of Fifth Avenue had been reduced, and as the last improvement had been long agitated and discussed, it was necessary to dispose of the question for all time, so as to permit property owners to develop the district with assurance of permanence of conditions.

The greater part of the area in question lies between Sixth and Fourth Avenues, and between Smithfield and Ross Streets, including Sixth, Oliver, and Fifth Avenues, and Diamond Street running east and west, and Cherry Way, Smithfield, Grant, and Ross Streets running north and south. The improvement was carried eastward as far as the intersection of Sixth and Fifth Avenues, and northward so as to afford a connection with Grant Boulevard, involving in this district Webster and Wylie Avenues, Strawberry Way, and the extension of Grant Boulevard. Oliver Avenue, Cherry Way, Diamond Street, and Fifth Avenue between Grant Street and Sixth Avenue were widened. The maximum cut of 16 ft. was at the intersection of Fifth and Wylie Avenues. The cut at Fifth Avenue and Grant Street was 14 ft., and this improvement is sometimes referred to as the 14-ft. "Hump Cut". As a result of the improvement, the grade of Fifth Avenue was reduced from 7.6 to 4.87% between Grant and Smithfield Streets, and the grades of the other streets in the district reduced in proportion. Fig. 2 shows the new and old grades of Fifth Avenue between Smithfield and Grant Streets.

The city bore the physical cost of the work, including the grading, reconstructing the entire sewerage system, relaying of water lines, constructing new pavements and sidewalks, tearing down buildings where the street lines had been changed, and similar work. Public service corporations at their own expense relaid street railway tracks, electric conduits, and gas lines. At the same time these companies generally enlarged their facilities. The property

owners attended to the shoring, underpinning, and protection of their buildings, and all the resulting reconstruction, alteration, and demolition work. Many of the older buildings were torn down, but a considerable number were lowered or additional stories provided below the former ground floor.

The city secured waivers of damage from nearly 80% of the property owners affected in so far as these related to the reduction of grade. The property owners, therefore, to a great extent defrayed the cost of the alteration and adjustment of their properties. The city, however, recompensed property owners for the net damages incurred due to widening of streets, this amount being materially reduced owing to the benefits accruing.

The total cost of the improvement has amounted to $2 364 673 and is itemized, as follows:

Cost of physical work—grading, regrading, paving, repaving, etc., Fifth Avenue, Diamond Street, and adjacent streets...	$732 019	
Cost of physical work—lowering water lines.	30 981	
Cost of alterations in Fire and Police Wiring System	13 771	
Cost of alterations to Public Safety Building	1 421	
Cost of repaving Sixth Avenue.....	2 772	$780 964
Damage awards on account of above improvements		499 059
Damage awards on account of widening:		
Fifth Avenue	$259 830	
Cherry Way	202 157	
Oliver Avenue	308 253	
Strawberry Way	130 403	
Grant Boulevard	315 404	1 216 047
Engineering and incidental expense.....		34 533
Total cost		$2 530 603
Benefits assessed by viewers:		
Widening Fifth Avenue.....	$595	
" Cherry Way	48 460	
" Oliver Avenue	92 875	
" Strawberry Way	14 000	
" Grant Boulevard	10 000	165 930
Net cost to City.....		$2 364 673

In addition, the expense borne by the Public Service Corporations and by property owners undoubtedly reached a very high figure.

The principal items of work and the unit prices were as follows:

Excavation, 153 900 cu. yd.....	@ $3.00
Block stone pavement, 13 800 sq. yd.....	@ 3.75
Creosoted wood block pavement, 11 000 sq. yd.....	@ 4.00
Vitrified brick pavement, 4 100 sq. yd.....	@ 3.00
Protected concrete curb, 17 200 lin. ft.....	@ 1.00
Concrete sidewalks, 149 700 sq. ft.....	@ 0.20
Lumber in temporary structures 27.3 M. ft. B. M.....	@ 80.00

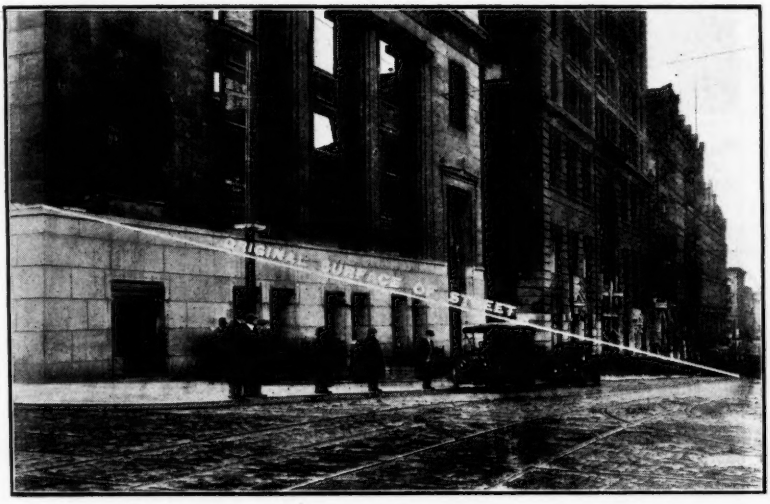


FIG. 2.—HUMP DISTRICT, PITTSBURGH, PA., VIEW OF FIFTH AVENUE, SHOWING NEW AND OLD GRADES.

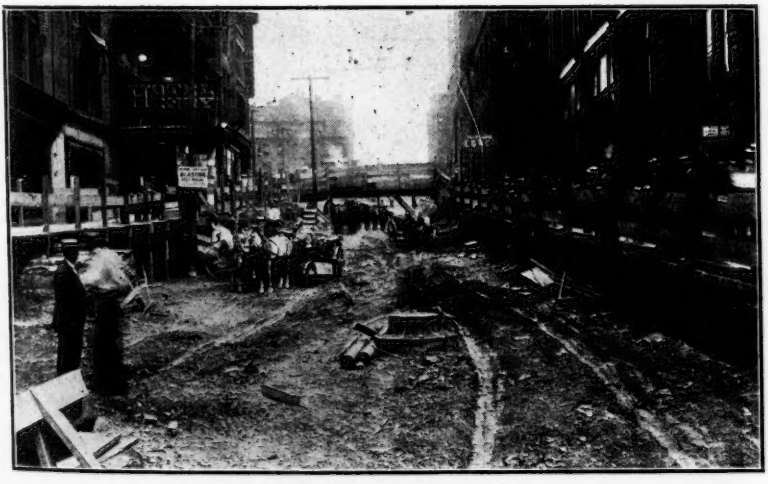


FIG. 3.—HUMP DISTRICT, PITTSBURGH, PA., FIFTH AVENUE FROM SMITHFIELD STREET TO GRANT STREET.



FIG.



F



FIG. 4.—VIEW SHOWING FLOOD OF MARCH 28, 1913, PITTSBURGH, PA., AT 30.4-FOOT STAGE, DUQUESNE WAY AND SEVENTH STREET, WEST.

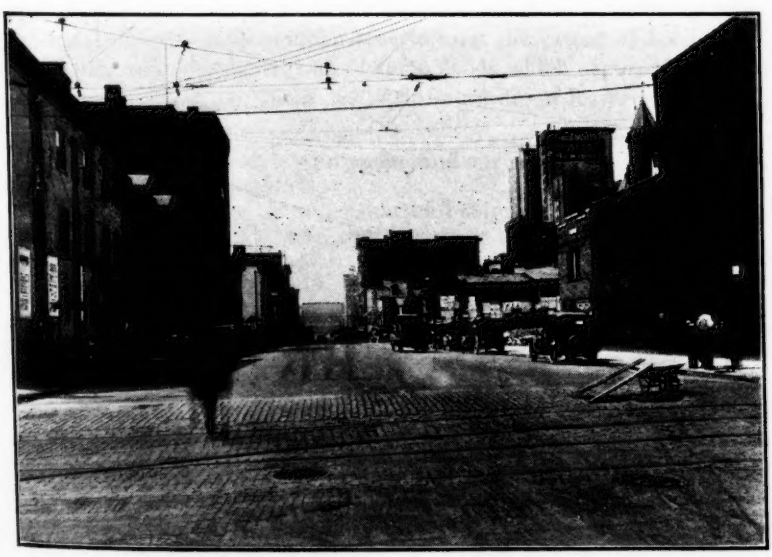


FIG. 5.—PITTSBURGH, PA., VIEW OF SECOND AVENUE WEST FROM GRANT STREET.





Work was begun April 5, 1912, and finished in November, 1913. The work was arranged so as to permit of the maintenance of traffic; work on Fifth, Oliver, and Webster Avenues was first completed, through traffic being carried on Sixth Avenue and Diamond Street. Pedestrian traffic was maintained by the erection of temporary boardwalks supported on trestles, and the public utilities were supplied by lines laid generally on the surface of the ground, as shown on Fig. 3.

Since the completion of the improvement, many costly structures have been erected within this area, among which are the William Penn Hotel, City-County Building, Union Arcade, Bell Telephone Company, Philadelphia Company, Harry Davis Theater, *Gazette-Times* Building, and others. The taxable valuation of properties in this district has greatly increased due in large measure to the erection of buildings on sites formerly vacant, or occupied by small structures. The increase in the taxation receipts for the district, assignable in a large measure to this improvement, has covered the cost and expenses borne by the City. In 1914, the Second Ward in which this district lies had an assessed valuation for taxable purposes, of $183 398 440 and, in 1924, it was $221 804 410, a gain of about 21 per cent.

Raising of Flood District

One of the problems with which the City of Pittsburgh has had to contend, and which has been remedied to a great extent, has been the periodic flooding of the low-level business, residential, and industrial territory along the rivers, causing serious and extensive damage to property, unsanitary conditions, and the suspension of business and traffic. River control and flood prevention have been extensively studied and reported on by the Flood Commission of Pittsburgh. This Commission, in ascertaining the extent of flood damages to the city, in a careful examination of three floods which occurred within a year, from March 15, 1907, to March 30, 1908, found direct losses within the city to be as follows:

Damage to buildings, equipment and machinery...	$782 400
Damage to materials.....	1 698 900
Loss to employers by supension of business.....	1 974 200
Loss to employees due to shutdown.....	1 308 300
Expense of cleaning up.....	547 400
Charities dispensed and funds for prevention of disease	27 800
Fires uncontrolled through inaccessibility or lack of water pressure.....	175 000
Total	$6 514 000

It was found that the loss ranges as follows:

For the flood of 27.3 ft.....	$414 700
" " " " 30.7 ".....	839 800
" " " " 35.5 ".....	5 259 500

The general condition of streets in the down-town, low-level district in 1913, during a flood stage of 30.4 ft., is shown on Fig. 4; and the enormous damage to property for floods of greater height can be easily surmised.

Since 1908, there have been fifteen floods recorded over the 25-ft. stage, and twenty-six times the Ohio River has passed the danger line of 22 ft.

The lower part of the down-town business section comprising an area of 60 acres, was raised to an elevation of 35 ft. and more, in order to avert this periodic flooding of streets. This re-grading of the low-level business district was started on April 28, and completed on December 1, 1915. The cost of the work, exclusive of the cost to the various utility corporations, was $162 000. The net damage to property owners along the improvement amounted to $165 000. The work was arranged to permit the maintenance of traffic and business. The property owners attended to the alteration and raising of their buildings to the new grades.

Having attended to the re-grading of its business area, immediately following the World War, the City began again to pay attention to the congestion of traffic in the down-town section. On July 8, 1919, a $20 000 000 bond issue was approved, and, as a result, the down-town section was materially re-arranged and the city in general improved. New avenues of traffic were provided; dangerous grade intersections eliminated; the water-works systems extended and improved; new highway bridges were constructed and some present bridges reconstructed; and provisions made for the improvement of parks and recreation facilities; extensions to the drainage system; and improvements for public health, welfare, safety, and charity services. Last, but not least, a sum of $6 000 000 was favorably voted for a start of subway construction in the down-town section.

The streets in the down-town section were materially re-arranged. Second Avenue from Grant Street to Liberty Avenue, was widened from 40 to 80 ft., having a roadway of 56 ft., sufficient for six lines of traffic, as shown on Fig. 5. Ferry Street from Water Street to Liberty Avenue, was widened from 40 to 70 ft. Diamond Street from Smithfield Street to Grant Street was widened from 25 to 54.5 ft., and Diamond Street from Ferry Street to Market Place was widened from 50 to 70 ft.

The widenings in the down-town section were very costly. The physical work in widening Second Avenue cost $177 733, and the net damages assessed against the City amounted to $1 851 460, making a net total cost to the City of $2 029 193.

The cost of the physical work on Ferry Street was $62 997, and the net damages assessed against the City were $666 480, making a total cost to the City of $729 479. The cost of the physical work on Diamond Street between Smithfield Street and Grant Street, only one block long, was $14 517, and the net damages assessed against the City were $391 795, making a total cost to the City of $406 312. On account of the valuation of property in the congested business section, it is quite evident that the cost of future widenings of streets therein will be prohibitive.

The cost in 1920 per square foot of net assessment against the City for widening Second Avenue was $45, for Ferry Street, $30, and for Diamond Street, $82 per sq. ft. The more important widenings of streets in the down-

town business section are shown on Fig. 6. Diamond Street, from Liberty Avenue to Ross Street, was widened in sections at three different times, and, as a result, there is now a zigzag alignment of street.

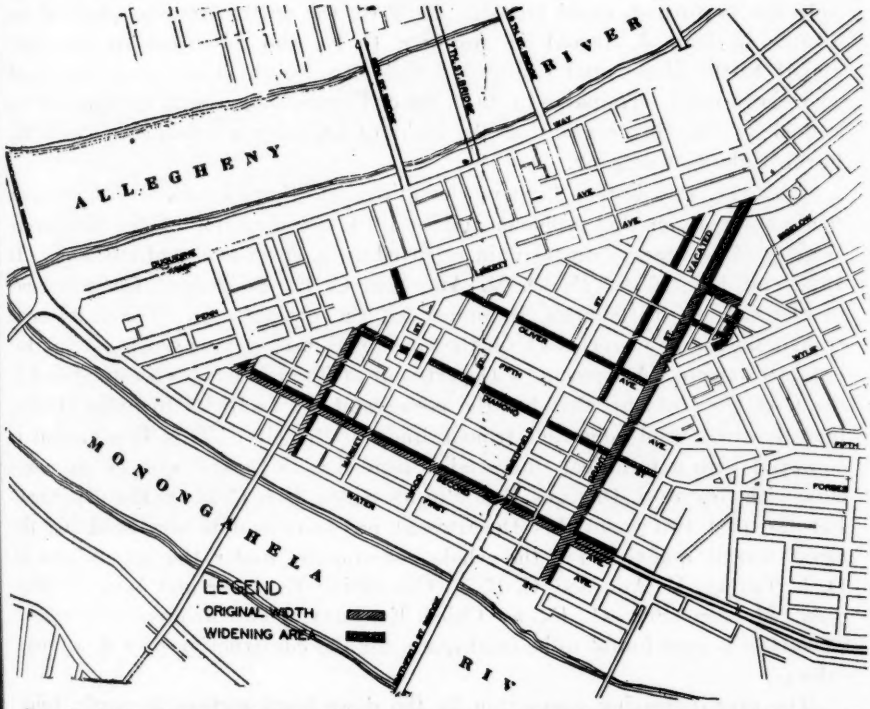


FIG. 6.—MAIN STREETS WHICH WERE WIDENED IN DOWN-TOWN BUSINESS SECTION OF PITTSBURGH, PA. DIAMOND STREET WHICH WAS WIDENED AT THREE DIFFERENT TIMES, IS NOW A ZIGZAG THOROUGHFARE AS TO ALIGNMENT.

More than any other single improvement, the new Boulevard of the Allies, caused the greatest re-arrangement of the street plan in this district. The Boulevard starts at Grant Street and Second Avenue, and extends eastward along the hillside to Bluff Street, by an elevated structure of easy grade. This Boulevard is a main thoroughfare to the down-town section from the east. It is one of the through routes of the city and has been chosen as a part of the Lincoln Highway through the city. Its location high on the hillside overlooking the Monongahela River as shown on Fig. 7, furnishes a picturesque driveway of a character distinctive of the rugged topography of the city. This thoroughfare which cost $2 600 000, was named Boulevard of the Allies as a memorial to the World War, and bronze tablets on columns at the entrance, as shown on Fig. 8, serve in commemoration of that event.

The item of a down-town subway at an estimated cost of $6 000 000 was the largest and the most important improvement proposed affecting the welfare and development of practically the entire community, especially as it opened up again the entire question of transit facilities for the City of Pittsburgh. The proposition itself was not even then new, and had been

considered by both the City and by private interests. As early as 1906, the subway project was under consideration by the City Council, when the Pittsburgh Subway Company was seeking a franchise for the building of subways to furnish rapid transit. In 1910, the entire traction problem was studied by Bion J. Arnold, M. Am. Soc. C. E., who submitted an exhaustive report to the Mayor and Council of the City, in which reference was made to a down-town terminal. In 1913, an ordinance was passed by Council but vetoed by the Mayor granting the right of building a subway to the Pittsburgh Subway Company.

In 1914, the Rapid Transit Committee of the Chamber of Commerce made a report on this problem, and in 1916, a Committee of the Engineers' Society of Western Pennsylvania reported on a down-town subway loop. In 1917 and 1918 E. K. Morse, M. Am. Soc. C. E., as Transit Commissioner of the City of Pittsburgh, recommended the construction of certain rapid transit lines. However, work on the construction of a down-town subway has not been begun. At present, a Department of Transit has been established in the City Government, that has the power and the facilities to make studies, investigations, and construct transit lines in the city. This Department is equipped with a technical staff, and is headed by a Commission of six members, with George S. Davison, Past-President Am. Soc. C. E., as the Chairman. For the past two years this Department has been and is now studying the rapid transit question for the whole community, under the supervision of D. L. Turner, M. Am. Soc. C. E., as Consulting Engineer and Winters Haydock, M. Am. Soc. C. E., as Chief Engineer. The original amount of $6 000 000 is now found to be inadequate for the construction of a down-town subway.

The ever-increasing congestion in the down-town section is partly being taken care of by parking restrictions, the establishment of one-way thoroughfares, and the re-routing of street cars. At present, most of the streets in this district are one-way thoroughfares, and the street cars are making use of short loops in the business district.

The City of Pittsburgh will again resume improvements to its public works, for in May, 1926, the people voted favorably for a bond issue of $19 902 000 to be expended for improvements to the various facilities and activities of the city. A summary of the proposed improvements is as follows:

For maintenance of service of existing facilities of water supply, bridges, sewerage, and streets.....	$7 950 000
For equipment and buildings for Fire and Police Departments to provide adequate protection.....	600 000
For the care of the City's poor, insane, and sick.....	2 750 000
For adding to the inadequate facilities of playgrounds.	750 000
For street improvements in widening and improvement of existing main thoroughfares to remedy traffic congestion and provide necessary arterial streets....	7 852 000
Total	$19 902 000

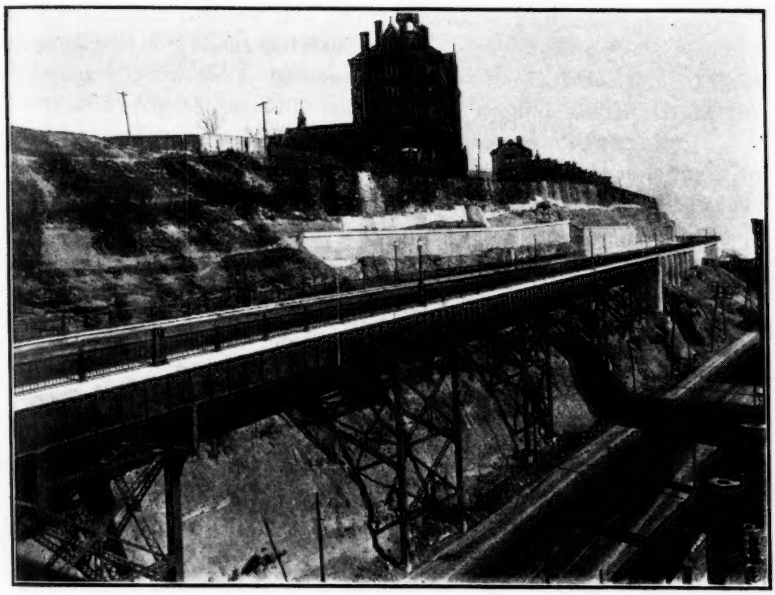


FIG. 7.—BOULEVARD OF THE ALLIES, PITTSBURGH, PA., VIADUCT NO. 1, VIEW EAST OF EAST END FROM ROE BUILDING.

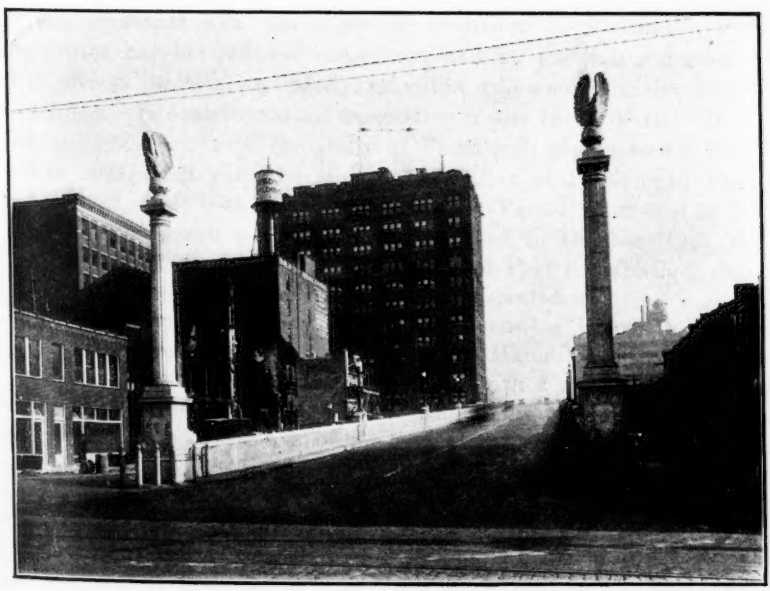


FIG. 8.—BOULEVARD OF THE ALLIES, WEST ENTRANCE, VIEW EAST, PITTSBURGH, PA.



Under this new bond issue Grant Street from Water Street to Seventh Avenue, will be widened 20 ft. on the easterly side, making an 80-ft. street. The total estimated cost of this improvement is $1 620 000, of which the physical work will cost about $90 000.

The damages estimated to be assessed against the City for this 20-ft. strip of property is $1 530 000, or $100 per sq. ft. of ground taken. Grant Street may be said to be the key street to the down-town section of the city.

The problem of transportation in the down-town section has brought forth the study of the development of the wharves of the city. A sum of $50 000 has been appropriated by Council for the payment of:

"Costs and expenses of making surveys, investigations, estimates, and plans for the improvement for public uses and purposes of the river-fronts of the Allegheny, Monongahela, and Ohio Rivers within the City of Pittsburgh, for providing adequate and improved facilities and terminals for all forms of river traffic and for such changes and improvements in the street, sewerage and drainage systems of the City, and of properties affected thereby, as may be necessary to carry out the foregoing improvements and for such other changes, improvements, and measures as may be deemed necessary therefor or desirable in connection therewith."

This study is now under way in both the Department of Public Works and the Department of City Planning, and it is expected that, as a result, the City will be again asked to vote on the bond issue to make possible the improvement of the wharves in the down-town section of the city.

Citizens Committee on City Planning

In 1918, the Citizens Committee on City Plan, an unofficial body of private citizens, was organized with the object of producing a "Pittsburgh Plan". This Committee has no political connection and no partisan purposes. It planned to give to the city, an orderly, scientific, and comprehensive program of city building. It contributed an exhaustive report in six parts: Part 1, published in 1920, dealt with the study of Pittsburgh playgrounds; Part 2, published in 1921, dealt with the Major Street Plan of Pittsburgh; Part 3, published in 1923, dealt with the transit question; Part 4, published in 1923, dealt with the Pittsburgh parks; Part 5, published in October, 1923, dealt with the railroads of the Pittsburgh District; and Part 6, published also in October, 1923, dealt with the problem of water transportation.

The City Planning Commission, with Morris Knowles, M. Am. Soc. C. E., as its Chairman, supplemented the report of the Major Street Plan as proposed by the Citizens Committee on City Plan, with a report on the Inter-District Traffic Circuit, issued in March, 1926. This report deals with the planning and estimate of the cost of the re-arrangement of the business district, and includes those thoroughfares, bridges, and other improvements that are, individually, integral parts of a so-called "Triangle By-Pass System", improvements of parts of tributary major radial streets, and modifications of minor streets immediately related to and affected by the changes proposed. The aggregate cost of these improvements will be more than $50 000 000.

In conclusion, it may be stated that the re-arrangement of a business district, such as that of the "Golden Triangle in Pittsburgh", is a basic problem of means of transportation, especially of the street plan.

The street plan requires careful consideration not only by the municipal authorities, but also by the various civic and trade organizations, citizens committees, and planning bodies, and includes problems of river and rail transportation, flooding of rivers, terminals, rapid transit, adjustment of grades, regulation of traffic, and zoning laws.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE PLANNING OF THE INDUSTRIAL CITY OF LONGVIEW, WASHINGTON*

BY S. HERBERT HARE,† ESQ.

"City Planning", a term applied to the orderly physical development of a community, would in most cases be more properly classed as "City Re-Planning".

The opportunity of planning a complete new city of any great extent on undeveloped land seldom presents itself. The planning of Longview, Wash., was perhaps the largest project of this kind since the planning of the National Capitol by L'Enfant under President Washington, and involved some pioneering in the field of city planning.

HISTORY OF THE DEVELOPMENT

A very brief historical account of the project is as follows: The Long-Bell Lumber Company, in looking forward to a continuation of its operations after its Southern timber supply was exhausted, purchased a large tract of timber land in Lewis and Cowlitz Counties, Washington, about 15 miles north of the Columbia River. In looking about for a mill site for the manufacture of this timber, a location on the Columbia River at the mouth of the Cowlitz was finally decided upon, and options were taken on land to cover the mill site and other lands in the valley. As the operations of the Company were to be very extensive and the number of people to be employed necessitated a great increase in the population of the district, the development of a town site adjoining the mills was discussed early in 1922. The purpose was not so much to secure the profits which might result from increased land values produced by the Company's operations, as to prevent exploitation of the land in the district and a haphazard growth of unrelated development in the vicinity of the mills. It was decided, therefore, that an industrial city should be built

NOTE.—Written discussion on this paper will be closed in December, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

* Presented at the meeting of the City Planning Division, Seattle, Wash., July 15, 1926.

† Landscape Archt. (Hare & Hare), Kansas City, Mo.

which would serve not only the Long-Bell interests but other industries as well. It was not to be a "company town" or a "one-industry town", and it was to be sold as there was demand.

In February, 1922, officials of the Company visited the site and sought advice on the feasibility of developing a town. This visit was followed later in 1922 by conferences, after which various industrial cities were investigated for information about developments similar to the one proposed, including land values, store counts, rail and transportation facilities, and many other subjects. In the meantime, a topographical survey of the lands involved was being made under the direction of Wesley Vandercook, Chief Engineer of the Long-Bell Lumber Company.

In June, 1922, after a thorough inspection of the property, the Company exercised options on approximately 14 000 acres of land, the area which has since been included in the plan for the city. At this time the writer made studies for both a skeleton plan of the entire area and for the detailed development of the interior section. In connection with these studies he had the general advice of the late George E. Kessler, Landscape Architect, and B. L. Lambeth, Realtor.

Upon arriving at a general scheme further studies were developed. These plans were made at different scales; the general plan at a scale of 1 000 ft. to 1 in.; plans for sections of the city at a scale of 400 ft. to 1 in.; and detail plans, each covering an area of 1 500 by 2 000 ft., at a scale of 40 ft. to 1 in. The detail plans, which showed lot lines, curbs, walks, easements, building and outbuilding lines, and other features, with controlling dimensions, were sent to Mr. Vandercook, in whose department calculations were made, streets and lots staked out, construction work supervised, and utilities planned.

After the initial planning was completed, special advice on commercial water-front treatment and railway problems was secured. As the building of the city progressed scores of detail plans were prepared for park areas, school sites, hospital property, and other public and semi-public developments. The first lot in Longview was sold in February, 1923, and the city was incorporated in February, 1924.

The Topography of the Site

The town site is principally a rich, flat valley bounded on the north by hills, on the south and southwest by the Columbia River, and on the east by the Cowlitz River. It includes more than 7 miles of deep-water frontage on the Columbia River, available for ocean-going vessels, and about 5 miles of frontage on the Cowlitz River suitable for barge service. On the north, the land included in the town site reaches into the hills at some points for a distance of about 1 mile and rises to an elevation of about 400 ft. The valley ranges from a little above tide level at the western end to Elevation 20 or higher at the point chosen for the center of the city. The land in the valley was included in seven separate diking districts which were merged into one large district, the intermediate dikes were removed, and a new and higher dike was built along the river-front.

The outstanding topographical feature of the valley is a small mountain rising from the valley floor near the western end. This mountain is more than 2 miles long and nearly 1 mile wide, reaching an elevation of 560 ft. at its peak. In some places it is very precipitous, covered with heavy tree growth, and in others more rolling on the upper levels with cleared farms. It is probable that the greater part of this interesting topographical feature, known as Mt. Solo, will be preserved as a large public park.

Another topographical feature which influenced the plan was a slough or old bed of the Cowlitz River which formed a loop about the place chosen for the center of the city. About 1½ miles of this has been preserved as a crescent-shaped parkway.

Adjoining the town site on the east is the Town of Kelso, Wash., which is situated on both sides of the Cowlitz River, and which had a population of 2 500 at the time operations at Longview were begun. Across the Columbia River on the south is the smaller town of Ranier, Ore.

The Plan

The plan of the city shows a system of main thoroughfares, or major streets, designed to carry the bulk of the through traffic, with secondary and minor streets to serve local needs. The main thoroughfares in some cases form radials, leading traffic to and from the center, but with ample by-passes both near and farther away from the center to prevent traffic being forced to the center. At the center of the radials is a 6-acre park known as Jefferson Square about which the public buildings will be grouped (Fig. 1). Fifteenth Avenue, which with Oregon Way, forms the main thoroughfare from the Columbia River to the business center, was purposely planned to be one block from Jefferson Square so that business frontage on this street would be continuous on both sides. Thus, although Jefferson Square is the focal point of the diagonal streets, it will not be the center of business activity. This business center is, however, as readily accessible by streets leading from the diagonals as the focal point itself.

The individual areas formed by the main thoroughfares and diagonals are subdivided for the most part in a rectangular or grid-iron manner with some irregular arrangements where the topography or the form of the areas dictated. The arrangement is, therefore, for the most part a systematic one so that streets can be easily named and numbered. The plan has been compared in a general way to that of Washington, D. C., perhaps because of the diagonals. It is, however, much more simple and there is an important difference in the treatment of the diagonals in relation to the rectangular streets. In Washington, the diagonals were superimposed upon a rectangular or grid-iron arrangement with no considerations given to the confusing angles which the intersections produced. At Longview, the rectangular arrangement is in a series of separate units, with connecting streets articulated across the diagonals, usually at right angles.

In order to clear the wholesale or light-industrial district to the southeast of the business center from the confusion which would be caused by a

diagonal, the diagonal in this direction was offset along the line of Fifteenth Avenue to a point where it could be used for an approach from the Pacific Highway across the Cowlitz River from the southeast. The location of this diagonal also offered an unusual opportunity for convenient railroad leads into the industrial districts, with only one track across the thoroughfare and with obtuse angles on the curves in the industrial tracts. The city is served directly by the Longview, Portland, and Northern Railroad, a common carrier, 30 miles long, which connects with the transcontinental lines just across the Cowlitz River.

After careful consideration a lot depth of 120 ft. was adopted as the standard for the regular portions of the city, as this depth was considered most suitable for the various uses. Alleys were provided quite generally, even in the residence districts, partly owing to a local custom in the use of slab wood for fuel. These alleys will probably not be provided in future high-class residence districts, and the districts which are being developed in the hills are irregular and carefully adjusted to the topography. A width of 20 ft. was adopted for the alleys, providing for two lines of traffic and pole lines.

In the street system, the main thoroughfares are 100 and 120 ft. wide, secondary thoroughfares and main business streets, 80 ft. wide, and other streets, 50 to 60 ft. wide. Roadways on the streets vary from 24 to 25 ft. on the minor streets to 60 ft. on the major streets. In some cases esplanades are used which may be removed when traffic demands. Block lengths are from 300 to 500 ft. in the business district and 600 to 1 100 ft. in the residence districts, the longer blocks being usually in the direction of traffic.

The distribution of schools has been given careful study and 5 acres or more have been reserved for grade school sites, with an area of 30 acres for a Junior and Senior High School, the latter to have an athletic field. Church sites, a golf club, cemetery site, and other community needs have been provided for as the complete plan developed.

In addition to the 6-acre park at the civic center other local parks and playgrounds have been reserved at convenient intervals. It is contemplated to retain some of the more precipitous slopes in the north hills for park lands, as well as the land on Mt. Solo previously mentioned, and some of the frontage on the Cowlitz River.

The development of a part of the slough surrounding the central section of the city (Fig. 2) promises to be one of the most interesting parkways in the country. The area reserved, containing 100 acres, is from 500 to 800 ft. in width, bordered by boulevards, and is 1½ miles long. To secure material for filling in the vicinity nearly 2 000 000 cu. yd. of sand was pumped by hydraulic dredge from this parkway, forming a series of connected lakes carefully designed to simulate natural bodies of water. Four thoroughfares will cross these lakes on bridges. Complete detailed grading plans were prepared for this development, not only showing the grading by sections, but accurately indicating by contours of finished grades the naturalistic rolling ground forms desired. This parkway has been named Lake Sacajawea, after the woman Indian guide of the Lewis and Clark Expedition.

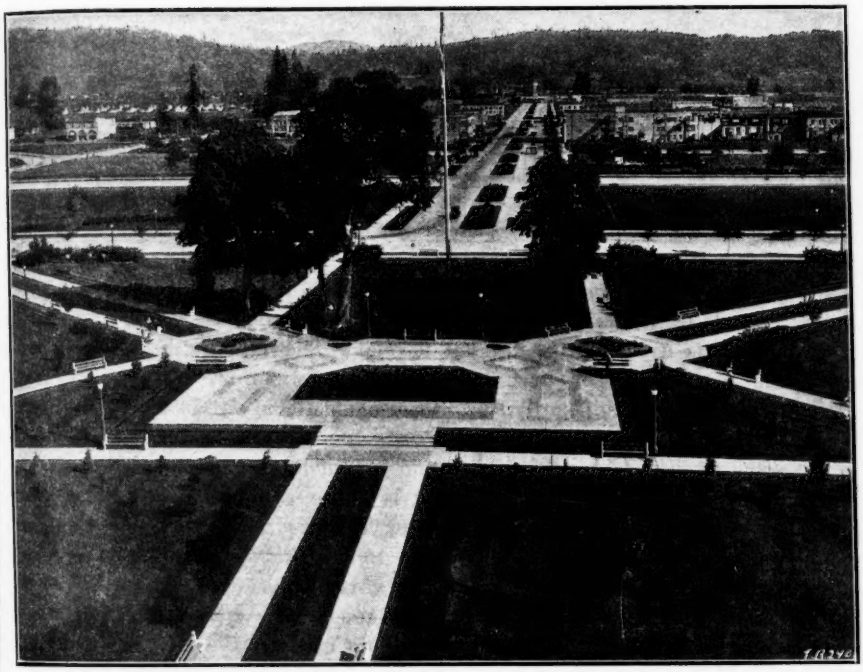


FIG. 1.—VIEW OF LONGVIEW, WASH., EAST FROM MONTICELLO HOTEL.



FIG. 2.—VIEW DURING DREDGING OPERATIONS ON LAKE SACAJAWEA PARKWAY, LONGVIEW, WASH. SMALLER HOUSE DISTRICT IN FOREGROUND.



Zoning

Zoning was one of the interesting problems of the planning of the city. The uses to which various properties were to be put was necessarily determined in the planning of the street system, and this street arrangement will tend to stabilize those uses and prevent shifting values. Early in the planning, a zoning map was prepared showing areas to be used for residences, apartments, business, and light and heavy industries. As there was no municipal corporation at that time to enact a zoning ordinance and no State enabling act to authorize such an ordinance, the zoning regulations were put into effect by private restrictions. These restrictions are partly in the deeds, but are mostly filed with the plat. They follow the most modern practice and are self-perpetuating, thorough, and complete, but in no way drastic, protecting the investment of the small home owner as well as that of the more wealthy one.

One of the difficult problems in connection with zoning was the necessity of starting development in a series of more or less disconnected centers rather than in one center. This at first produced a scattered appearance and extra cost of paving and utilities, but the wisdom of the procedure is already apparent as the population is growing, and the scattered centers are gradually being joined into a homogeneous whole without conversion of property from one use to another. The plan provides for a population of 50 000 and by converting suburban acreage into city lots a much larger population could be housed on the town site. The installation of utilities and the problems of diking and drainage have been intricate.

Throughout the planning it was constantly kept in mind that this city was to be a thoroughly practical, general industrial city, one in which convenience and economy, although of primary importance, were to be combined with the highest type of beauty—that beauty which is designed into a development as an organic part rather than added to it as an ornament. The people of Longview will work amid pleasant and convenient surroundings and will enjoy some of the amenities of life.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

MODERN CRUSHED STONE AND GRAVEL SURFACING*

BY J. W. HOOVER,† ESQ.

In his address at the Washington Convention of the National Sand and Gravel Association, on January 25, 1923, E. W. James, Chief of the Division of Design of the United States Bureau of Public Roads, stated that, of the roads constructed with the assistance of Federal Aid funds, 40.4% of the mileage was gravel surfaced and that such roads had absorbed approximately one-fourth of the Federal Aid funds expended to that date. States, counties, and townships construct independently, or without Federal assistance, several times the road mileage built with Federal aid. The subject, therefore, is of considerable importance to the Nation and to each individual community.

Much might be said of the economic value of stone and gravel surfacing; of the volume of traffic that will justify the change from a dirt road to a gravel or crushed rock surface and from such surfaces to more expensive and satisfactory pavement. The time at which each change becomes advisable must be determined for each road. To state as a general proposition that a traffic of 300 or 400 cars per day, or so many tons per day, justifies a change of type of surfacing, is to err. The cost of maintenance of the existing surface and the expense of the construction and maintenance of a higher type of surface differ for each locality. The value for a highway in the State of Washington would not serve in another State except as a coincidence; the principle involved is the same, however, and should be based on the various costs as affected by sub-grade traffic, and climate.

The most important consideration is service to the public. Just how good a road is the public entitled to have? As a general rule, the automobile owner is supplying the funds; it is his money and his road; how good should it be?

A number of engineers state that the paving of a highway is warranted when the traffic has reached 400 to 500 cars per day, or approximately the same number of tons per day based on certain average conditions in certain

NOTE.—Written discussion on this paper will be closed in **December, 1927.**
* Presented at the meeting of the Highway Division, Seattle, Wash., July 15, 1926.
† State Highway Engr., Olympia, Wash.

localities. None has attempted to fix the volume of traffic that would be considered sufficient to justify gravel surfacing; the general idea seems to be that if a road is worth building at all, a gravel surface is justified. The writer doubts if many paved roads are carrying less than 1 000 cars per day, and thousands of miles of gravel roads are carrying more than 1 500 cars per day.

At present, the question of the time to change the type of surface is dependent on the funds available and seldom on the real economics of the situation for the reason that improvements are far behind the requirements of traffic. A great majority of the highways are now carrying more traffic than the width and type of surface justify, considering the economic value and the service to which the public feels entitled. The question of which road shall be improved first may be answered on the basis of traffic, although often the decision is made by the legislative body making the appropriations, disregarding the merits of the respective roads.

The sub-grade on which the surfacing materials are to be placed should provide a well-drained stable foundation. The soil, as well as practical drainage, and the climate should be considered before the surfacing materials are selected and before the width and thickness of surface are determined.

After the sub-grade has been studied and the present and probable future traffic has been determined, the metal—either gravel, crushed gravel, or crushed rock—may be selected. The criterion for such selection should be that the material selected furnish the service required at the least cost. Hand in hand with the selection of the metal is the design of the cross-section.

A surface 12 ft. wide and 6 in. thick may be all that is necessary for light traffic. However, there is, and no doubt will be for some time, a large mileage of unpaved highway that is subjected to traffic sufficient to justify paving. The writer would recommend a width of not less than 18 ft., a thickness at the edges of 9 to 10 in., and a center thickness of 7 to 8 in. If the sub-grade is unstable, additional thickness should be provided. A design similar to that described is of more importance where water-bound macadam and dust-prevention methods through oiling or other surface treatments are contemplated or probable in the near future.

At first, the maximum size of gravel or crushed rock was rather large, often as much as 3 in. With more improved and extensive maintenance methods, the size was reduced until 1½ in. for the base course and ¾ in. for the top course became common, with occasional specifications limiting the base rock to a maximum of 1 in. and the top course to ¾ in.

Practically all these roads were maintained by frequent dragging, and the ideal was expressed as a smooth compact base covered with ½ in. to 1 in. of loose material to be worked back and forth across the road by the drag or grader. Highway surface construction passed through the coarse rock stage and there is now a large mileage of fine rock surfacing that may be treated to give a surface free from dust.

A tendency is now developing to change to coarser rock or gravel in surface treatments in order to secure a smooth, compact surface free from loose material where the travel is heavy and where dust prevention is desirable.

The selection of gravel, crushed gravel, or crushed rock as a surfacing has been based largely on the cost of the material on the road. This has resulted in natural gravel surfaces or crushed gravel surfaces in the glaciated areas and crushed rock surfaces in unglaciated areas. In general, crushed rock has given the best service, with crushed gravel and pit-run, screened gravel in the order named. Deposits of natural fine gravel, however, sometimes give as good results as the best crushed rock.

The question of hardness of gravel or rock for surfacing has been studied a great deal and, in general, the limits set forth in the specifications of the various States produce good results. It is not practical to use the same specifications in all the States on account of differences in the materials available, the soil, traffic, and climate. Very often, the material available would not pass the specified tests, nevertheless, it makes the best of road surfacing under the circumstances. The policy of the State of Washington is to test the materials before the contract is advertised. Through the specifications the bidders are then advised of the requirements for each grade of material. In this way the contractor is fully advised before making his bid as to the sources of gravel or rock, uncertainty is greatly reduced, and the State receives a better bid for the work.

The following is a brief review of gravel or crushed-rock surfacing specifications for a roadway that is to be maintained by dragging loose material back and forth:

(1) The sub-grade shall be smooth and true to the required cross-section and all large rock removed. The surfacing material shall be uniformly distributed on the sub-grade.

If the inspector should be lax in this requirement the surfacing will not be smooth and traffic will work depressions where the material is not uniform and considerable time may be required before the maintenance forces can furnish a first-class riding surface. The usual specifications also require that the contractor, as far as practical, shall do his hauling over the surfacing as it is placed.

(2) The surfacing material shall be dragged as much as necessary to keep it smooth. Where the surfacing is to be in two courses, the contractor is required to construct as much of the base course as possible from any one setting of bunkers before beginning the top course. If the entire product of the crusher is to be placed in the base course, the entire base course shall be constructed before starting the top course. If the product of the crusher is separated into two sizes only, the base course is constructed until the section of the bunkers for the finer material is filled, at which time the top course may be started.

In this way the hauling equipment is used to secure as much compacting of the two courses as practical. By dragging the surfacing and keeping it smooth the trucks do not follow the same tracks, and the traffic is distributed over a wider area. Few specifications give sufficient attention to the binding material for the surfacing. The average specification permits the use of side-of-the-road material, or any other material that is easily available, that has binding qualities. The binding material should be a water-resistant clay

which will harden when mixed with the surfacing material. The State of Oregon has given this part of the work a great deal of attention during the past few years and has secured good results.

Too much emphasis cannot be placed on the necessity for keeping the sub-grade and each course of surfacing smooth during the entire construction of the road and afterward by the maintenance organization. The best results are obtained in this type of surfacing by constructing it in at least two courses, the base course containing rock of a maximum size of 1½ in. and the top course composed of rock which will pass a ¾-in. ring. The two courses should be of equal thickness, and each not less than 3 in. The thickness of the surfacing depends on the character of the sub-grade, the traffic, and the climatic conditions. In some places, such as the arid regions of Washington where the annual rainfall is only 6 to 10 in., light surfacing wears very well. In other parts of the State a very heavy surface course is necessary as the annual rainfall may be more than 100 in. per year and the sub-grade very soft when wet. The best results in the heavy rainfall area are obtained by using the product of the crusher in the base course as this forms a crust with comparatively little binder due to the interlocking effect of the crushed material. In the arid regions the product of the crusher may be separated into the two sizes, the base course being bound with clay after which the top course is placed and bound with clay. A great variety of specifications for the size of material in each course is possible, dependent on the materials that are available.

The available materials should be thoroughly studied for each project. In some localities there is no choice. Surfacing in more than two courses is often advisable on account of a soft sub-grade that cannot be drained or raised without excessive cost. In some instances a base course of large rock is advisable and economical.

Gravel or crushed rock surfaces are maintained by dragging with a grader or drag having a comparatively long wheel-base. Some roads must be dragged more often than others, depending on the stability of the sub-grade, the character of the surface, the kind and volume of traffic, and the climatic conditions. Additional material must be added as the original material is ground up and blown away or pounded into the sub-grade. The surface must be kept smooth, for if it once becomes rough a considerable cost is entailed in scarifying, re-shaping, and compacting.

Surface treatment with oils gives promise of eliminating the dust nuisance. In some States oil treatment will be costly, as the type of surfacing in use is not adapted to such treatment and is less expensive than surfacing on which oil works best.

There is no final answer—no one final specification for a modern crushed stone or gravel road; possibly there are but few fundamentals that will apply under all circumstances. Soil, climate, traffic, materials of construction, and funds fix the limits to which construction may be carried. Surfacing of the type discussed is being studied and will be improved. Highway engineering has made wonderful strides in the last ten years and gives promise of producing a road surface that will serve adequately in many places until higher types of pavement are warranted.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

RELATION OF ROAD TYPE TO TIRE WEAR*

By O. L. Waller† and H. E. Phelps,‡ Members, Am. Soc. C. E.

Synopsis

The study of the wear of automobile tires has been made to determine the destructive effect of different types of road surfaces on the tread rubber in order that, as far as possible, surfaces may be so improved as to make the cost of building and maintaining road surfaces and of operating the traffic over them a minimum. The cost of tires is one element of the cost of operating motor vehicles and may well be studied.

The University of Kansas, co-operating with the United States Bureau of Public Roads, and the State College of Washington have each conducted a series of tests in an effort to determine the loss of tread rubber caused by several types of road surfaces. The surfaces studied have been concrete, brick, and bitulithic pavements, penetration macadam, and several types of gravel and broken stone. The results of these tests are given in this paper. The factors influencing tire wear are discussed, and an effort is made to evaluate some of these factors. The following tentative conclusions have been drawn from the few data now available:

1.—Tire wear increases with the speed. Sufficient data are not available to indicate the rate of increase.
2.—Tire wear increases with the temperature. Road tests indicate this relationship, but are not consistent and yield no information on the rate of increase. Laboratory tests by Professor W. C. McNown, at the University of Kansas, are likewise not conclusive, but indicate a moderate rate of increase in tire wear with increase of temperature. High temperatures are very destructive to tires.
3.—The wear of rear tires is greater than that of front tires, the relative wear of rear tires being 200% on smooth pavements and as little as 118% on gravel surfaces, probably averaging 150% of the wear on front tires.

Note.—Written discussion on this paper will be closed in December, 1927.
* Presented at the meeting of the Highway Division, Seattle, Wash., July 15, 1926.
† Vice-Pres., State Coll. of Washington, and Head, Dept. of Civ. Eng., Pullman, Wash.
‡ Prof., Highway Eng., State Coll. of Washington, Pullman, Wash.

4.—Tire wear per ton of car is probably constant for any given road surface, when size of tire, load, and inflation pressure are determined by a single standard.

5.—A few records of bus companies show actual tire-mileage life on different road surfaces in approximate agreement with tire wear tests on similar surfaces.

6.—Relative tire wear index numbers, based on average concrete pavement as 1.0, are approximately 2.0 for good gravel and macadam, 4.0 to 5.0 for average Western macadam or uncrushed chert gravels, and may be as high as 10 or 11 for unusually unfavorable surfaces. Based on tire wear alone, average sized cord tires should show a life of 20 000 to 24 000 miles on pavements, 10 000 to 12 000 miles on good gravel or macadam, and as little as 2 000 to 4 000 miles on unfavorable surfaces. The corresponding costs per vehicle-mile are about 0.5 cent for pavements, 1.0 cent for average gravel or best macadam, 2.3 cents for average macadam, and perhaps as much as 5 cents per vehicle-mile on road surfaces of loose, sharp-edged stones.

Introduction

The making and operating of automobiles and the building of roads on which they are run are major American industries. In 1925 automobile owners paid $260 619 621 for registration fees, licenses, fines, etc., and, in addition, a gasoline tax, effective in all but four States, totaling $146 028 940 or a total payment by automobile owners and operators of $406 648 561 almost all of which was expended for highway purposes.

The total funds expended in the construction and maintenance of roads and streets is about $1 000 000 000 annually. About 10 000 miles of Federal Aid roads were completed in 1925 and perhaps 20 000 miles of surfaced roads were built by States and counties without aid from the National Government.

Such an enormous investment in motor vehicles and highways makes the problem of securing the most economic surface for highway transport of the utmost importance. This problem will be solved when, for any particular highway, the road surface as well as the vehicles using the road are such as to produce minimum transportation costs.

In making economic comparisons between different road surfaces, costs which arise from the operation of the vehicle and those which arise from the highway itself, must be considered in order that the total cost of highway transportation may be a minimum. The condition is rapidly being reached in which the automobile owner pays both the cost of operating his car and a large part of the cost of building and maintaining the roadways upon which it operates. He should be interested, therefore, in the effect of various roadway surfaces on motor-vehicle operating costs as well as in the cost of building and maintaining such surfaces.

Road money cannot all be spent on a few miles of exceptionally fine arterial highway. Such a plan would favor some and leave the less fortunate travelers to use unimproved dirt and mud roads.

If all roads could be paved, a great saving in operating expenses would be made, but this is not possible. Only roads carrying heavy traffic can be paved.

On those carrying lighter traffic, gravel and broken stone surfaces must be used. The other millions of miles of plain dirt roads are usually not well-graded nor well drained; but traffic which uses these poor roads must pay not only its share for constructing the paved roads, but also must then use the unimproved roads with the added expense per car-mile. The vehicle-mile should be made the measure of road expenditures. This is a measure of the traffic service rendered.

The cost of operating a motor vehicle is the sum of the costs of gasoline, oil, tires, maintenance, depreciation, interest, insurance, garage, and license, the first five being items influenced by the roadway surface on which the vehicle is generally operated.

The gasoline requirements of motor vehicles on various roadway surfaces has been investigated by T. R. Agg, M. Am. Soc. C. E., and others, and their findings are generally known and frequently quoted. It is generally believed that the maintenance and depreciation costs are likewise influenced by the character of the road surface over which the motor vehicle is most frequently operated, but no determination of the magnitude of this influence has yet been made to the knowledge of the writers. Tire wear tests have been made at the University of Kansas and the State College of Washington. These tests are brought together in Table 1, and summarized in Tables 2 and 3. The results summarized in Table 3 are given in two units, as tire wear in pounds per 1000 miles and as wear on various surfaces relative to the wear on concrete pavements taken as a standard.

In making tests to determine the wear on tread rubber a considerable number of elements enter into the problem, some of which are as follows:

Factors Affecting Tire Wear.—

I.—Those pertaining to the road surface:
- (*a*) The kind of surface.
- (*b*) Its condition.

II.—Those pertaining to the car:
- (*a*) Weight of car:
  - (1) Sprung.
  - (2) Unsprung.
- (*b*) Number of engine cylinders.
- (*c*) Character of springs.
- (*d*) Alignment of wheels.

III.—Those pertaining to the driver:
- (*a*) The speed used and wind resistance.
- (*b*) The method of operating brakes.
- (*c*) Number of emergency stops.
- (*d*) Skids, acceleration, and deceleration.
- (*e*) General care and skill displayed in driving.

IV.—Climatic conditions:
- (*a*) Air temperatures.
- (*b*) Amount and distribution of rainfall.
- (*c*) Amount of sunshine.

V.—Those inherent in the tire:

(*a*) Kind of tire.
(*b*) Quality of the tire.
(*c*) Character of the tread.
(*d*) Inflation pressure.
(*e*) Temperature of the tire
(*f*) Age of the tire.

In order to study the effect of any one factor on the wear of tires all other factors should be kept constant during a single series of tests. This may be accomplished by using the same equipment and by making the tests over the same piece of road in the same condition of maintenance and under corresponding temperature conditions at the same speed, with tires of the same type and age and with the same degree of inflation, with the same driver and with the chassis in the same state of maintenance. In this way a considerable number of the unknown factors may be eliminated. There will still remain the type of road, the condition of the surface, both as to roughness, sharpness of material, compactness, and the quantity of uncompacted loose material floating over the surface.

The relationship between tire wear and gasoline consumption might profitably be studied. If such a relationship can be determined then every one will have a reasonably good gauge by which to measure the destructive effect of different types of road surfaces on tires and car upkeep. To do this, the gasoline should be metered by a recording device to show graphically the gasoline consumption as a continuous record.

The relationship between surface roughness and gasoline consumption and tire wear would supply much needed information on road surface types and on the best methods of maintaining earth and gravel roads. A continuous autographic record of the gas metered to the carburetor should be kept and made to synchronize with a graph showing vertical uneveness.

Preliminary to the tests at the State College of Washington, the tires and rims were removed from the wheels and were thoroughly cleaned with a brush and bellows. The valve cores were then removed and the deflated tires with the rims were accurately weighed. The valve cores were then replaced and the tires inflated to the proper pressure and replaced on the wheels. The car was then driven at uniform speed over the selected road, and at the conclusion of the test the tires were again removed, cleaned, and weighed.

The roads used for test purposes were of brick, concrete, bitulithic, penetration macadam, gravel, and Western type broken stone macadam. All the tests on brick roads were made in Kansas by Professor McNown, who likewise conducted tests on concrete pavements, penetration macadam, and gravel roads made of several different materials. The State College of Washington tests were made on bitulithic and concrete pavements, water-worn gravel, and Western type broken stone macadam.

Crushed Stone Roads in Washington

The crushed stone highways of Washington, owing to lack of water in many localities, are not water-bound. Water has been available for only a few roads. The general practice is to use finer material than is common in water-

TABLE 1.—RESULTS OF TESTS.

No. of test.	Average loss, in grammes per 500 miles.	Loss, in pounds per 1 000 miles.	Kind of road surface.	Remarks.
R- 1	47.25	0.2083	Concrete	Aggregate, ⅔ soft limestone; ⅓ Joplin flint; surface rough.
R- 3	20.50	0.0904	"	Aggregate, medium limestone; surface very good.
R- 8	37.25	0.1642	"	Aggregate, medium limestone; surface very good.
R-13	48.50	0.2128	"	Aggregate, medium limestone; surface very good.
R-16	83.25	0.3571	"	Surface fair; balloon tires.
R- 5	61.00	0.2690	Brick-monolithic	Surface good.
R- 9	48.25	0.2117	Brick-bituminous filled	Air, 92°.
R-10	29.00	0.1279	" " "	Air, 92°.
R-14	53.50	0.2359	" " "	Balloon tires; air, 64°.
R-15	11.00	0.0485	" " "	Air, 90°.
R- 4	68.00	0.2998	Bituminous macadam	Left-hand wheels.
R- 4	282.00	1.2334	" "	Right-hand wheels.
R-11	283.25	1.2389	" "	.....
R-12	301.50	1.3294	" "	.....
R- 2	148.00	0.6526	Gravel	Gravel from uncrushed chert.
R- 7	60.50	0.2668	"	Typical of good gravel; loose surface, unplaned.
36	.....	0.0325	Concrete	Surface good, but not excellent, Car 1.
38	.....	0.0629	"	" " " " " " 1.
37	.....	0.0760	"	" " " " " " 2.
39	.....	0.0360	"	Surface good, but not excellent, (1 tire only) Car 2.
42	.....	0.1136	"	Surface generally good, Car 3.
48	.....	0.0960	"	" " " " 3.
43	.....	0.2080	"	" " " " 4.
49	.....	0.0570	"	" " " " 4.
44	.....	0.1020	Bitulithic	Car 3.
46	.....	0.1890	"	" 3.
45	.....	0.1170	"	" 4.
47	.....	0.0910	"	" 4.
50	.....	0.723	Gravel	Water-worn, Car 3.
52	.....	0.527	"	" " 3.
1	.....	0.542	Macadam	Pullman to Palouse, Car 1.
5	.....	0.271	"	" " " " 1.
7	.....	0.360	"	" " " " 1.
19	.....	0.1418	"	" " " " 1.
21	.....	0.443	"	" " " " 1.
23	.....	0.275	"	" " " " 1.
25	.....	0.343	"	" " " " 1.
31	.....	0.290	"	Dishman south, " 1.
40	.....	0.244	"	" " " 1.
8	.....	0.590	"	Pullman to Spokane, " 2.
35	.....	0.618	"	Dishman south, " 2.
41	.....	0.777	"	" " " 2.
11	.....	1.122	.....	Pullman south, " 2.
17	.....	1.045	.....	" " " 2.
27	.....	1.456	.....	" " " 2
33	.....	0.435	.....	" " " 2.
1	.....	0.911	.....	Pullman to Palouse, " 3.
4	.....	0.271	.....	" " " " 3.
6	.....	0.573	.....	" " " " 3.
18	.....	0.965	.....	" " " " 3.
20	.....	0.528	.....	" " " " 3.
22	.....	0.697	.....	" " " " 3.
24	.....	0.453	.....	" " " " 3.
9	.....	0.308	.....	Pullman to Spokane, " 3.

bound macadam, both in the bottom and the top courses. After construction, the roads are left for the traffic to roll. The crushed stone used is generally basalt of good quality as to hardness, toughness, and resistance to abrasion. It is usually screened into two bins, one receiving the material from ⅜ in. to 1½ or 2 in. in size, and the other bin receiving the finer material.

TABLE 2.—SUMMARY OF TESTS.

	Loss, in pounds per 1 000 miles.
Best results on concrete, Washington	0.0325
" " " " , Kansas	0.0904
Average of all tests, Washington concrete	0.0852
" " " " , Kansas concrete	0.1689
Poorest results on Washington concrete	0.2080
" " " Kansas concrete, cord tires	0.2128
" " " Kansas concrete, balloon tires	0.3571
Monolithic brick, a single run	0.2690
Bitiminous-filled brick, average	0.1560
Bitulithic, average	0.1247
Bituminous macadam, best	0.2998
Bituminous macadam, poorest, average	1.2672
Gravel, typical good, Iowa	0.2668
Gravel, uncrushed chert	0.6526
Gravel, Washington, water-worn, average	0.6250
Macadam, Washington type, best	0.2440
" " " , average	0.5800
" " " , poorest	1.4560

The stone is spread in two courses, each approximately one-half the total thickness. A layer of clay or sandy loam is spread over the base course for a binder. The traffic and trucks hauling stone do the rolling, and a blade grader is frequently run over it to keep the ruts filled. Over this a finer top course is spread, but no binder is used. The surface of the road is left with a layer of sharp, fine, loose material, which adds considerable resistance to the movement of the car and to the wear on the tires.

TABLE 3.—RELATIVE TIRE WEAR ON VARIOUS ROAD SURFACES.

Character of surface.	Wear, in pounds per 1 000 miles.	Index number.
Bitulithic, average of all tests	0.1247	0.945
Concrete, average of all tests	0.1319	1.00
Brick, average of all tests	0.1786	1.36
Bituminous macadam, best	0.2998	2.28
" " , poorest	1.2672	9.60
Gravel, typical, Iowa	0.2668	2.02
Macadam, Washington type, best	0.2440	1.85
" " " , average	0.5800	4.40
Gravel, uncrushed, best	0.6526	4.95
Macadam, Washington type, poor	1.4560	11.05
Concrete, average of seven best	0.0657	0.49

MAINTENANCE

The broken stone and gravel roads of Washington are in all stages of maintenance. The surfaces of new roads are loose, just as they are left by the contractor after the top course had been trued up with the blade grader. Under such conditions, automobile tires push the loose gravel or broken stone out to

the sides of the road and form ruts, a process destructive to tread rubber and one that forces the car continually to ascend a constant incline.

The same conditions, only in a lesser degree, obtain on roads compacted through use that have been recently bladed to fill pot-holes and to plane off corrugations. This kind of maintenance leaves a thinner layer of loose material for the traffic to work over and through. Where maintenance is not frequent and regular, the roads soon develop pot-holes and corrugations, which greatly reduce riding comfort, shorten the life of the car, and increase the destruction of tread rubber.

For the broken rock and gravel roads here enumerated, it is difficult to describe the condition of the particular road surface under test, and certainly no standard surface conditions can be secured. The surface of any piece of test road will be subject to wide variations. Even on the same road two runs might give a wide variation in gasoline consumed and rubber worn away, depending on whether the car was driven over the well beaten track or on the loose, floating material.

It should be kept in mind that at normal speed and where the road surface is rough, full of pot-holes, or corrugated, tires are not in contact with the surface at all times, or at least the pressure on the road is not constant. When the rear wheels return to the road, the impact kick of the wheels tends further to destroy the road surface and abrade the tread. The tests of the U. S. Bureau of Public Roads have shown that the impact force in some instances has been equivalent to several times the static load. Under such conditions, the tread rubber, being always under tension, in transmitting such added power to the road, is under an additional stress, which adds to the destruction of the tread rubber.

There is generally little uniformity in these broken rock surfaces. Under wear, they range from hard smooth surfaces to loose gravel, corrugations, and pot-holes, and all these conditions may be found on any stretch of sample road.

Corrugations on the broken rock roads examined were about 26 to 27 in. from crest to crest. Consequently, the automobile wheels jump from the crest of one corrugation to the trough of the next, roll up the incline to the succeeding crown, and jump to the next trough. This continuous jumping and back-kicking of the rear wheels, wears down the tread rubber on the tires very much more rapidly than it is worn over pavement, well bound broken rock or graveled road.

In all tests, the comparative roughness of the surface of the road tested and the approximate size of the aggregate material should be considered. As measured by roughness, there are many kinds of broken rock surfaces and many kinds of rocks used in such surfaces. Some kinds of broken rock present sharp edges not found on water-worn gravel.

Some gravel and broken stone roads present well compacted and hard surfaces; on others, the gravel is loose, either from having been recently placed and therefore not well bonded, or from having been recently bladed and left with a floating surface of loose material. The grade of material used, and

the degree of "wash-boarding" or corrugation should be noted. Some standard of thickness of the loose floating material on the surface, and of the grade and sharpness of material, should be taken as a criterion, and all roads used in tests should be referred to this standard.

Some samples taken from a broken basalt road a year after the road was built showed the sharp edges practically gone and the fine material very much rounded. It would seem that the crushing and grinding effect of the wheels soon wears away sharp angles, rounds up the stone, and makes an old broken rock surface, when in good order, less destructive to tires than a new one.

Gas Consumption

Professor Agg states* that fuel consumption is not a direct measure of the power developed by a gasoline motor, but it is believed that the results thus obtained are comparative and sufficiently accurate for the purposes of this investigation. If the same car were used with the same carburetor setting, the same kind of gas, and operated by the same driver, the gas consumption curve would likely closely parallel the one for tire wear. Professor Agg states that "until more conclusive information pertaining to the variation of mileage costs other than gasoline is available, it is necessary to assume that all mileage costs vary with the resistance of the road surface in the same manner as does the gasoline cost." The investigations of the U. S. Bureau of Mines show that carburetor adjustments and the use or non-use of the air heater make a marked difference in mileage secured; consequently, if gas consumption is to be a reasonable measure of tire wear, all conditions relative to the car must be made uniform.

Yielding surfaces, due either to moisture or heat, require a greater consumption of fuel; more power is transmitted through the tires to the road, thereby causing a more rapid destruction of the tires.

Spring Action

It has long been known that spring action is a leading factor in the upkeep of a car. It is an important factor in the life of the car and as well as the life of the tires. Efficient springs damp the blows and thereby minimize the work done by the carcass of the tires and by the tread rubber.

Road Material

Theoretically, it would be expected that water-worn gravel would abrade a tire less than crushed rock, as crushed rock offers sharp edges to the tire. This seems to have been proved by the tests in the State of Washington on broken basalt roads and in Kansas, as shown by the tire wear, on crushed chert. The abrading effect of road materials should be studied further so that flinty material breaking with sharp cutting edges might be avoided. With such material, the more it is ground up by use, the more abrading angles it presents to rubber tires.

* *Bulletin 65*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa.

Tire Wear and Speed

It seems evident that tire wear should be influenced by the speed at which a car is operated. More power is required to drive a car at high speed than at low speed, and the bounding and spinning of wheels due to ruts should be more destructive of tread rubber at the higher speeds.

The few tests of tire wear made thus far at different speeds reveal no consistent relationship between these factors. The tests at different speeds made at the State College of Washington were on broken stone covered with a mulch of finely divided broken stone and under constant maintenance with blade graders. The differences in wear on the several tires of a car were considerable, and the various runs with the same car did not show closely similar tire wear. Plotting tire wear against speed for the cars operated on a single stretch of road produces a wide distribution of points, indicating roughly that tire wear increases with speed.

It seems obvious that high speeds should be very destructive of tires. It is well known that the life of a tire on a racing car is relatively short, although the destruction of such tires may not be due to the factor of tire wear. Of two similar tires, the one used on a car driven at moderate speed should travel many more miles than one on the same car driven at high speed.

Some light on this relation of tire wear and speed may be had from other investigations of closely related factors. The subject of power loss in automobile tires seems to have been first thoroughly investigated in the United States by Professor E. H. Lockwood, and has been continued by Messrs. W. L. Holt and P. L. Wormeley at the U. S. Bureau of Standards.

In a report on "Research on Rubber Tires" to the Advisory Board on Highway Research, Mr. Holt draws the conclusion, that,

> "The power loss [in automobile tires] is very nearly directly proportional to the speed, that is, at 40 miles per hour the loss is approximately twice that at 20 miles per hour. This means that the resistance which a tire offers to rolling, or what will be referred to as the rolling resistance, is very nearly constant and is independent of the speed".

The same result was reached much earlier by Professor Lockwood who reported that "one surprising result of all measurements of rolling resistance at speeds from 20 to 40 miles per hour has been the slight increase of resistance at the higher speed. With some cars the increase of resistance is less than the observation errors. In other cases the increases amount to from 5 to 10 per cent". Considering all tests, Professor Lockwood concluded that the internal rolling resistance of an automobile is practically constant at all speeds up to 40 miles per hour; and from other tests that the tires are responsible for nearly two-thirds of the power loss in the car itself.

Both these tests were made on electrical absorption dynamometers, the essential feature of which is a smooth, flat-faced metal drum on which the tires revolve. Although tests on such apparatus show that the rolling resistance of tires is constant at all speeds, as Professor Lockwood reports, "the rolling resistance on rough, uneven and soft roads will doubtless increase materially with the speed".

In all tests made on absorption dynamometers the wind resistance is automatically eliminated. This resistance must be overcome by the power of the engine transmitted to the surface of the road through the tires. This wind resistance has been shown by L. E. Conrad, M. Am. Soc. C. E., to be given by the formula, $R = 0.0025\ A\ V^2$, in which, A is the projected area of the car and V is the relative velocity, in miles per hour, of air and car. Since wind resistance causes an increased tractive force to be exerted it seems logical to conclude that tire wear will likewise increase with the speed.

Tire Wear and Temperature

In addition to increased wind resistance, higher speeds increase tire temperatures that may result in increased tire wear through the softening of the rubber, and lessen the mileage life of tires. The generation of heat and the resultant rise of temperature of tires are the result of the work done in overcoming the rolling resistance. This increase of temperature has been computed by Professor Lockwood, and checked almost exactly by a suitably conducted test.

A few tests have been made to determine the effect on tire wear of variations in air temperature, but as yet have not yielded consistent results. The road tests made in 1924 showed that tire wear increases with an increase of air temperature, the other variables of speed, road surface, car, and driver being kept constant.

The tire wear tests of 1925 show no consistent relationship between tire wear and temperature, probably because the tests on any one road extended over such a limited temperature range.

Laboratory tests by Professor McNown, at the University of Kansas, show that tire wear is increased at higher temperatures. The increased wear at 130° Fahr., as compared to the wear at 70° Fahr., amounted to 13% in one set of tests, and to 81% in another, the results not being entirely conclusive.

It is a popular impression that tires wear out more rapidly at high than at low temperatures. A dealer furnishing tires for a Washington bus line that descends a vertical distance of 2 000 ft. on an average 4.2% grade, ascribes the comparative low mileage (12 000) of the tires used on this run to the heat generated by the brakes on this grade. Tourists across the deserts of Southern Nevada and California describe the landscape as littered with abandoned tires, and believe the heat causes a rapid deterioration of tires.

Considerable trouble is being experienced by auto-bus companies with tires being damaged through heat. A heavy bus traveling on a fast schedule requires frequent application of the brakes, and the heat thereby generated produces destructive temperatures, particularly on the inside tire of a dual mounting. Experience and tests have both demonstrated that brake-drum temperatures of 550° to 600° Fahr. are common. Ordinary operation over heated pavements will produce temperatures of 150° to 160° Fahr. Casings and tubes will withstand these temperatures without trouble, but they will not withstand the much higher temperatures produced by a near-by brake-drum. Bus companies are in a position to force the dealers to supply tires on a

mileage guaranty, and the experience of many dealers has not been a pleasant or profitable one. Notwithstanding the short life of many tires damaged by heat, there is little information showing the exact relationship between the wear of a tire and its temperature.

A leading bus company operating in the State of Washington, has kept tire cost records by months. On one set of buses operated by this company, forty tires were used; on the other line, seventy-four. The buses make the same trip each day; the same number of trips each month; consequently, most of the unknown factors are eliminated. The plotted curve shows a very rapid increase as the season advances to midsummer, and a similar rapid decrease as fall approaches. On one bus line the average cost for November, December, January, and February, was about one-third that for July and August. The distance traveled was 148 104 miles per year. On the other line, the July-August average was more than six times the November-December-January-February average. The distance run was 105 690 miles per year.

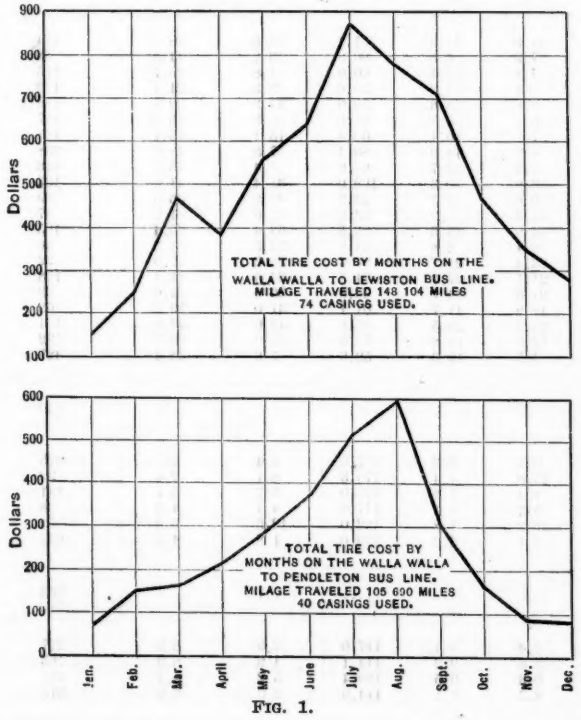


FIG. 1.

The loads may have been heavier and the roads somewhat rougher in the dry summer months than in the winter. This may account in part for the wide variation in tire costs between the summer and winter months. Fig. 1 shows these tire costs by the month.

Relative Tire Wear on Front and Rear Wheels

It is generally believed that tires wear more rapidly on rear than on front wheels. Tests show very erratic results, but averages support the general belief. The tests made at the State College of Washington show average rear tire wear of 140% on macadam, 158% on gravel, and 200% on pavement, as compared to front tire wear. The Kansas tests show smaller differences, the averages being 118% for gravel and 138% for concrete. Professor McNown states that normal front tire wear is 50% that of rear tire wear, but this may not be correct for all road surfaces.

TABLE 4.—Relative Wear, Front and Rear Wheels.

Test No.	Average Wear, in Grammes.		Miles per test.	Average Wear, in Grammes per 100 Miles.		Percentage of wear on rear tire in terms of wear on front tire.	Surface.
	Front.	Rear.		Front.	Rear.		
State College of Washington Tests.							
2	11.3	17.3	61.1	18.5	28.4	154	Macadam
5	7.9	6.8	60.0	13.2	11.3	86	"
7	7.1	12.4	60.0	11.8	20.7	175	"
19	9.1	12.9	59.6	15.3	21.7	142	"
21	8.6	13.6	60.0	14.3	22.7	159	"
23	6.7	9.8	61.3	10.9	16.0	147	"
25	7.9	10.1	60.4	13.1	15.7	128	"
34	6.3	14.9	80.4	7.8	18.6	238	"
40	5.2	12.7	80.9	6.4	15.7	246	"
8	58.0	70.3	169.0	31.4	41.6	133	"
11	9.6	8.5	23.3	41.2	36.5	89	"
17	11.3	9.9	23.6	48.0	41.9	87	"
27	14.9	15.6	23.5	63.4	66.4	105	"
33	6.1	2.8	24.9	24.5	11.3	46	"
35	23.4	21.7	80.4	29.2	27.0	92	"
1	21.1	27.1	61.1	34.6	44.4	128	"
4	21.6	14.7	60.0	36.0	24.6	68	"
6	18.9	11.7	61.1	31.0	19.2	62	"
18	25.4	25.5	59.6	42.6	42.8	101	"
20	12.0	15.8	60.0	20.0	26.3	132	"
24	4.5	19.3	59.6	7.6	34.4	427	"
Average	....	....	.....	....	....	140	Macadam
36	0.7	3.9	157.0	0.4	2.5	625	Concrete
37	13.8	4.4	170.3	8.1	2.6	32	"
42	4.4	7.9	120.0	3.7	6.7	181	"
48	5.2	5.1	117.8	4.4	4.3	98	"
43	15.2	7.8	122.0	12.5	6.4	51	"
49	2.1	5.0	119.0	1.8	4.2	233	"
Average	....	....	.....	....	....	203	Concrete
44	3.4	7.4	117.0	2.9	6.3	217	Bitulithic
46	5.6	9.2	114.1	4.9	8.0	163	"
.....	6.3	6.5	120.4	5.2	5.4	104	"
.....	2.3	7.1	114.9	2.0	6.2	310	"
Average	....	....	.....	....	....	198	Bitulithic
52	9.2	14.5	58.7	....	....	158	Gravel
Average	....	....	.....	....	....	158	Gravel

TABLE 4.—(*Continued*).

KANSAS RESULTS

Test No.	AVERAGE TIRE WEAR, IN GRAMMES PER 500 MILES.		Percentage of wear on rear tires in terms of wear on front tires.	Surface.
	Front.	Rear.		
R- 1	32½	62	191	Concrete.
R- 3	11½	29½	256	"
R- 8	53½	21	39	" Same as R-3.
R-13	59	38	64	" " " "
R-16	81½	85	104	" Surface fair.
Average	....	....	131	Concrete.
R- 5	94	28	30	Brick—Monolithic.
R- 9	64½	32	50	" – Bituminous filled.
R-10	40	19	47	" " "
R-14	59	48	81	" " "
R-15	15	9	60	" " "
Average	....	....	54	Brick.
R-11	289	278	93	Bituminous macadam.
R-12	290	314	108	" "
Average	....	....	100	Bituminous macadam.
R- 7	60	61	102	Gravel. typical Iowa.
R- 2	136	160	118	Gravel, uncrushed chert.

Professor McNown also states that,

"It is to be expected that rear tire indices would be higher for the rough surfaces due to the bounding and spinning of rear wheels. There is abundant evidence in the above data, however, to indicate that for rough surfaces under the condition of this test the front and rear tire wear is more nearly the same than for well conditioned surfaces".

The tests at the State College of Washington (Table 4) abundantly bear out this statement.

These tests were made on Western type broken stone macadam, which, when well maintained, is covered by a thin mulch of finely broken rock, and when newly constructed or in poor condition, by a considerably thicker layer of ¾-in. rock. On such surfaces the front wheels are being pushed through a layer of loose material, spreading it out and preparing a path for the rear wheels, with the result that tire wear is more nearly equal on front and rear wheels operated over such surfaces. On smooth pavements the front wheels merely roll over the surface, the rear wheels propelling the car. The result should be that on pavements the relative wear of rear tires would be greater than on macadam.

Front tire wear is frequently greatly increased by worn bushing or other parts that throw the front wheels out of alignment. The writer knows of one case where two front tires were entirely ruined by less than 1 000 miles

of travel on account of misalignment of the front wheels. In all the tests quoted, attention was given to front wheel alignment. The extremely erratic nature of the results indicate that many additional data must be secured before dependable conclusions may be reached.

Effect of Weight of Car

Owing to a lack of sufficient data an analysis of tire wear as influenced by the weight of the car is not entirely conclusive. The results (Table 5) seem to show that tire wear per ton of vehicle is either independent of the weight of the vehicle, or increases slightly with an increase in the weight of the vehicle. Averaging all results on concrete pavements, the tire wear per 1 000 miles per ton of car was 9.8% greater on the heaviest car than on the lightest, the increase of weight of car being 153 per cent. Although all four cars were not tested on the same pavement, tire wear increased with increased weight of car for the two pavements tested, and for all four cars tested.

TABLE 5.—Summary of Tire Wear Tests. Effect of Weight of Car.

Car No.	Weight, in pounds.	Average wear, in pounds per 1 000 miles.	Average wear, in pounds per 1 000 miles per ton of car.	Kind of road surface.
1	1 740	0.0477	0.0549	Concrete, Spokane to Coeur d'Alene.
2	2 750	0.0760	0.0552	" " " " "
3	3 560	0.1048	0.0589	" Olympia to Vancouver.
4	4 400	0.1335	0.0603	" " " "
1	1 740	0.349	0.401	Macadam, Pullman to Palouse.
3	3 560	0.628	0.353	" " " "
1	1 740	0.353	0.407	" average of all runs.
2	2 750	0.863	0.623	" " " " "
3	3 560	0.588	0.330	" " " " "
1	.....	.....	0.407	" " " " "
2 and 3	.....	.....	0.479	" " " " "

On broken stone macadam the results are not consistent. Of the two cars tested on the same road between Pullman and Palouse, Wash., the light car had more wear per ton than the heavier. The average of all tests on macadam are erratic, inasmuch as the medium weight car shows the greatest tire wear per ton. This may possibly be accounted for by the fact that this model of car is notoriously known to be very rough riding and hard on tires. The average tire wear on the two heavier vehicles is 18% greater per ton of car than on the lighter one, the increase of weight being 81 per cent.

It is probable, therefore, that tire wear is independent of the weight of car, at least for those cars supplied with the correct size of tire and designed by the same engineering standards. There is some evidence to show that the wear on an overloaded tire is greater than on one carrying a more reasonable load.

Effect of Road Surface on Life of Tires

Although such tests as have been reported yield valuable information on the effect of various road surfaces on tire wear, the real factor desired is the effect of road surfaces on the mileage life of tires. This can be determined only by a long-time test, starting with new tires and operating the car con-

tinuously over the same road surface until the tires are destroyed. Many such tests would be necessary in order to secure dependable averages, and the cost of such testing would be considerable. The mileage of tires is sometimes greatly reduced by such factors as tread cuts, blisters, stone and rail bruises, shimmying, under-inflation, punctures, corrugations, ice, ruts, pot-holes, etc.

Probably the most reliable method of determining the mileage life of tires on various road surfaces, would be to secure the life history of tires from the owners of cars operated exclusively on certain road surfaces. This is difficult, as few owners keep accurate records of their tire mileage and fewer still operate their cars exclusively on one kind of surface. Cars operating exclusively on surfaces with similar wear characteristics are probably mostly owned by taxicab companies and automobile bus lines. A few records from Western bus lines and other sources are given in Table 6. These records were kept as a part of the cost data and presumably are fairly accurate.

In general, these scattered records are in agreement with the tire wear tests, inasmuch as tires operated over pavements have a mileage life at least twice as great as those driven over gravel or macadam.

In addition to a greater tire mileage on pavement, bus companies report other items of expense materially greater when operating on gravel or macadam surfaces. One company reports that there is much more tire trouble on earth, gravel, or macadam surfaces. Tacks and nails are more easily picked up and are more frequently the cause of punctures. Cuts in casings fill quickly with dirt, which soon causes blisters and tread separation.

The life of a tire is generally not determined by external but by internal wear. An interesting confirmation of this fact was obtained from Mr. Lewis A. Corbett, of Walla Walla, Wash., who stated that on the bus line to Lewiston, Idaho, tires were seldom discarded from excessive wear of tread rubber, but generally as a result of internal defects, whereas on the line to Pendleton, Ore., tires frequently lasted 30 000 to 40 000 miles. Mr. W. L. Holt, Mechanical Engineer of the U. S. Bureau of Standards, has stated that:

"Contrary to what seems to be the popular idea, the problem in tire building is not to make a tread which will wear, but to make a tire in which the component parts—the plies, the breaker, the tread, etc.—will stay together, and at the same time distribute the various strains so that the strain will not be excessive at any point. As soon as any of the parts separate, chafing immediately sets in, blisters begin to appear at various places, and the life of the tire is about gone".

Professor McNown, however, has stated that "tire manufacturers are of the opinion that at present the carcass is developed ahead of the tread and that the normal failure of a tire is by tread wear and not by carcass failure". Probably this is true for present-day tires running over improved surfaces, but it may not be true for unimproved surfaces where stresses on the carcass are very much greater.

Tire Operating Costs

Tests of tire wear on various road surfaces will be of value both to the automobile owner and to public officials who build and maintain the highways.

TABLE 6.—TIRE OPERATING DATA FROM BUS COMPANIES.

Owner, or authority.	State	From:	To:	Kind of bus.	Size of tire, in inches.	Length of run, in miles.	Reported mileage.	Surface.
Red Ball Transportation Company...	Iowa	Mason City	Algona	Fageols Macks	.....	....	28 000 to 30 000	Concrete.
Red Ball Transportation Company...	Iowa	Mason City	Various places	Fageols Macks	.....	....	20 000	Earth and Gravel.
Hawley.....	Washington	Spokane	Coeur d' Alene	.....	36 by 6	33	30 000	Concrete.
Hawley.....	Washington	Spokane	Almira	.....	36 by 6	....	16 000	58 miles of macadam. 24 miles of concrete.
Hawley.....	Washington	Spokane	Almira	....	.....	....	Summer, 58 miles of macadam. 12 000 24 miles of concrete.	
Spokane-Lewiston Stage Line	Washington	Spokane	Lewiston, Idaho	.....	34 by 7	115	12 000	102 miles of macadam. 13 miles of concrete.
S. C. Hadden.....	Indiana	Indianapolis	Richmond	2½-Ton. G. M. C.	.....	....	28 000	Concrete.
Mrs. W. I. Coldiron.....	.....	Indianapolis	Bloomington	Studebaker	.....	....	12 000	Gravel.
J. C. Corbett.....	Washington	Walla Walla	Lewiston	White, two-ton trucks	.....	100	11 800	Macadam.
J. C. Corbett.....	Oregon	Walla Walla	Pendleton	White, two-ton trucks	.....	42	16 000	Bitulithic.

The automobile owner is chiefly interested in his tire cost per mile and for the year, whereas the highway engineer is interested in determining the amount he can afford to spend in providing road surfaces that reduce tire wear.

TABLE 7.

Road surface.	Index No.	Computed tire mileage.	Annual tire cost per car for 6000 miles per year.	Cost of tires per vehicle-mile.	Annual cost per miles of road per 100 cars per day.	Tire operating cost, capitalized at 5 per cent.
Concrete, average	1.0	22 750	$31.65	$0.00533	$190	$3 800
Bitulithic	0.95	24 000	30.10	0.00502	180	3 600
Brick, average of all	1.4	16 250	44.30	0.00738	266	5 300
Macadam, best	1.9	12 000	60.10	0.01002	361	7 200
Gravel, typical Iowa	2	11 375	63.30	0.01055	380	7 600
Bituminous macadam, best	2.3	10 000	72.80	0.01213	437	8 700
Macadam average	4.4	5 175	139.40	0.02323	835	16 700
Gravel, uncrushed chert	5.0	4 550	158.25	0.02637	950	19 000
Bituminous macadam, poor	9.6	2 375	302.00	0.05033	1 728	34 600
Macadam, loose	11.0	2 050	316.00	0.05267	2 090	41 800

The following analysis of tire operating costs is based upon the tire wear indexes of Table 3, which are tentative only and subject to change as more data became available. A standard 32 by 4-cord tire will be assumed, with 3 lb. of rubber available for abrasion during its life. The cost of such a tire will be taken as $30, with interest at 5 per cent. Based on these assumptions, Table 7 has been computed.

TABLE 8.—Passenger Automobile Costs.*

Composite car.	Cents per mile.
Gasoline	1.61
Oil	0.31
Tires	0.98
Maintenance	1.24
Depreciation	5.13
Interest	1.24
Insurance	0.31
Garage	0.83
License	0.59
Total	12.24

* *Bulletin 69*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa.

As a check on the cost of tires per vehicle-mile, Table 8 is presented. This table shows the cost per mile of various items of expense for a composite passenger automobile. The tire expense for this composite car is given as 0.98 cents per mile. This compares with the tire costs of practically 0.01 cent per vehicle-mile of Table 3 for typical Iowa gravel or the best Western type macadam.

The data used in this paper have been largely obtained from the following sources:

(1) *Proceedings,* Third, Fourth, and Fifth Annual Meetings, Highway Research Board, Report of Committee on the Economic Theory of Highway Improvement.
(2) *Bulletins Nos. 16* and *17,* Engineering Experiment Station, State College of Washington, Pullman, Wash.
(3) *Bulletins Nos. 64, 65,* and *69,* Engineering Experiment Station, Iowa State College, Ames, Iowa.
(4) *Technologic Papers No. 213* and *240,* U. S. Bureau of Standards.
(5) *Concrete Highway Magazine,* April, 1926.
(6) Michigan State Highway Department Investigation of Truck Performance on Grades.
(7) *Proceedings,* Tenth Annual Road School, Purdue University.
(8) "Power Losses in Pneumatic Tires", by E. H. Lockwood, *Transactions,* Society of Automotive Engineers, 1917.
(9) "Chassis Friction Losses", by E. H. Lockwood, *Transactions,* Society of Automotive Engineers, 1922.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

STREAM POLLUTION IN THE PACIFIC NORTHWEST*

By William F. Allison,† M. Am. Soc. C. E.

The problems presented by the pollution of streams and lakes in the States bordering on the North Pacific Ocean are different from those in older communities only in degree and acuteness. Much of the difficulty that has arisen is due to a feeling that streams and lakes flowing into the Pacific Ocean, and its arms, are unlimited in their capacity to absorb pollution, and that, therefore, population and industry need not seriously concern themselves in disposing of wastes which have little, if any, commercial value.

In the early days the chief form of pollution was caused by the miners who stirred the muck and gravel from the bottom of the streams, and after removing the gold, sluiced the tailings out on the arable land below as the easiest method of disposing of it. Usually, these arable lands were still in the hands of the Federal Government, but this was slight bar to the easiest disposal of waste as placer miners saw fit. The miner will still contend that prior use of stream and land gives him a permanent right to use them for his purposes, regardless of the interest of later comers. Much of the land covered by this waste had agricultural possibilities and was eventually acquired by private parties who, to secure their rights, carried the contest into the Courts. These lawsuits are particularly notable in California, where they have invariably been decided in favor of the farmer. The placer miner, when he stakes a claim, makes the possibility of disposal of tailings one of his chief considerations.

Agriculture and mining again come in conflict when the lode miner, after running his ore through a stamp mill, seeks a means of disposing of the tailings of the mills. This waste is usually pulverized quartz which,

Note.—Written discussion on this paper will be closed in December, 1927.

* Presented at the meeting of the Sanitary Engineering Division, Seattle, Wash., July 15, 1926.

† Prof., Municipal and Highway Eng., Univ. of Washington, Seattle, Wash. Prof. Allison died on July 6th, 1927.

when carried down to irrigation ditches and finally deposited on an alfalfa field, is highly injurious to the growing crops, acting not at all like the silt deposited in the Valley of the Nile during a period of flood.

The farmer, who must use the water of these streams to irrigate his land, must find some method of disposing of this finely pulverized quartz before it reaches that land. Ordinarily, this is done by impounding the water in a reservoir where the suspended particles are allowed to settle. It is periodically removed from the reservoir or settling basin at a cost of a great deal of hard labor and much time.

This sedimentation does not remove the solids in solution which may have a deleterious effect on this water for domestic purposes, and to some extent, for irrigation. Some of the streams of Colorado have an appreciable quantity of sulphuric acid which sedimentation and screening through gravel does not remove. The taste of the water is much impaired and its use in the making of tea and coffee is quite unsatisfactory, inasmuch as the tea takes on the color of ink.

The heavy deposits of pulverized quartz are now being cared for by compelling the mining and milling companies to dispose properly of the wastes due to their operations. However, this is done with some reluctance, as increased cost may result in suspension of work in a basic industry and a consequent unemployment of labor. The problem is similar to that, but not precisely the same as, in the mining regions of the East.

Lumbering is another basic industry in the Pacific Northwest that has a waste problem to solve. This problem is practically the same as that faced by the lumberman of the East. Sawdust and refuse from saw-mills are more easily disposed of by depositing them in a stream than in any other manner. There they rot and in a short time become exceedingly foul, to the destruction of fish, and thus they tend to destroy another basic industry. Quite generally this is overcome by compelling the saw-mill operators to burn their waste, which perhaps causes some danger to plant and surrounding timber. The much heralded danger from forest fires must be considered when plans for burning waste are considered.

A third growing industry that seriously pollutes the water of streams and lakes is the manufacture of wood pulp and paper. The sulphite liquor used in pulp making is harmful to fish life, and it is found that small fibers in the waste become entangled in the gills of fish and cause their death. An economical and satisfactory method of caring for this waste has not yet been found. The problem is an old one and any efforts to solve it should be made along lines which will not injure the pulp and paper business. Competition with European pulp is keen and any irksome restrictions may drive American manufacturers out of business.

The Federal Government is spending much money in the promotion of fisheries on the Pacific Coast. It cannot look with approval on pulp manufacture on the coast of Alaska, where there is said to be much suitable pulp wood, if pulp making destroys young fish which have been so carefully reared and protected.

The protection of fish is one of the chief objects of Fish and Game Commissions of the States of this region. Over-fishing and out-of-season fishing is prohibited, but, in any contest with other industries, the fish are likely to be sacrificed to the economic demands of other desirable lines of endeavor.

The effect of sewage pollution has a considerable bearing on the fishing industry of the Pacific Coast, as is common on the Atlantic seaboard. The shellfish industry of Puget Sound has been only partly developed, but already the presence of *B. coli* in the oysters has been observed. It is believed that oyster culture can be greatly increased if steps are taken to prevent raw sewage from flowing into Puget Sound. A proper educational program will be necessary to overcome prevailing ideas in this respect. The sale of oysters grown in certain localities on the Atlantic Coast is generally forbidden by Boards of Health and the same regulations may be anticipated regarding the oysters of the Pacific Coast if steps are not taken to prevent pollution of areas that may be used as oyster beds.

There is now a considerable business in the marketing of clams, but inasmuch as clams are almost always cooked before eating, it is felt that there is no great danger from contamination, although clams obtained from sludge deposits would be in little demand from the public and probably would not have the approval of the health authorities.

Sewage seems to be very destructive to fish life that has a high commercial value. A few years ago, near one of the salmon hatcheries in the southern part of the State of Washington, it was observed that young salmon on being turned into a stream near the mouth of a sewer, died very quickly. A considerable portion of the sewage flowing into the stream was traced to a hospital, and it was believed that there was enough poison in the sewage to kill the fish. Probably storm sewage containing oil and gasoline from the streets would have the same effect.

The two most conspicuous examples of sewage pollution of fresh water in this State are the disposal of the sewage of the City of Spokane in the Spokane River, and the pollution of Lake Washington at the eastern edge of the City of Seattle by the sewage of urban and suburban districts bordering on that lake. Spokane permits its sewage to go into the Spokane River, below the city, which, in turn, flows into the Columbia River.

It is contended that the Spokane River, in flowing over a series of falls and rapids and through ponds which act as settling basins, causes the sewage to oxidize to such an extent that it becomes practically harmless when it reaches the Columbia. It is by no means certain that this is true. Typhoid is more or less endemic all along the eastern slope of the Cascade Mountains and is sometimes a menace to the surrounding country. Water used for irrigation is often rather heavily polluted, and workers in the fields and on the fruit ranches are careless about drinking water from the irrigation ditches, being thus exposed to the fever. The occasional cases that are found in the cities of the Puget Sound region are contracted in this manner, and make it difficult to stamp out typhoid on the Pacific Coast. Regulations and rules of Departments of Health are frequently violated and, for several years the

Board of Health of Washington has carried on a campaign of education in matters relating to the pollution of streams and of the proper disposal of sewage. It has seemed that education of this kind must face the decided opposition of political and commercial interests.

For several years, the before-mentioned pollution of Lake Washington has been a subject of controversy. The lake is a natural receptacle for the drainage of a considerable area. This area lies partly within the city limits of Seattle and includes the rapidly growing district east of the lake. Until a few years ago the lake was the source of water supply for the city. With the growth of the city and the towns surrounding the lake, it became increasingly more difficult to prevent sewage from draining into the lake. Private sewers became a constant source of trouble. After Seattle began to obtain its water from Cedar Lake, forty miles southeast of the city, it was believed by people living near the lake that the difficulty was settled for all time. The best and most accessible fresh-water bathing beaches, however, are here available, and there is much sentiment against their destruction.

The construction of the ship canal and locks prevents for all time the use of lake water for domestic purposes, as the operation of the locks admits an appreciable quantity of salt water into the lake. It has been contended by some that a fresh-water harbor would not compensate the city for the loss of fresh water which, in the future, might be needed for a city supply. However, it seems probable that sufficient water may be obtained from mountain streams for all purposes, although it will probably have to be filtered and sterilized before using. Be this as it may, the flow of sewage into the lake has materially detracted from its value for bathing purposes, and the problem now presented is to treat the sewage in such a manner that bathing may be safe and satisfactory. Bonds have been voted to build intercepting sewers to carry the sewage of this district into Puget Sound, which is an expedient that will improve the water in the lake, but will tend to increase the trouble with the Sound waters. The writer can see no solution of the problem except to treat all sewage flowing into the Sound and to sterilize it to the satisfaction of the State Board of Health.

The City of Seattle has had some trouble because of the location of and opposition to small sewage treatment plants. A careful survey of surrounding property, together with the proper design and operation of such plants, perhaps might tend to reduce the opposition. Certainly, the benefit that may be derived from them should be much greater than the damage caused.

Water obtained from mountain streams is not always free from sewage, although the sources of such streams are springs and glaciers. The City of Wenatchee, Wash., proposed a few years ago to obtain its water from a glacial stream called the Icicle, hoping thus to eliminate the typhoid prevailing in and about that city. Upon examination of water from the Icicle, it was found that a considerable number of *B. coli* was present, and the water could not be approved by the Board of Health unless satisfactorily treated. This, together with the heavy cost of obtaining the water from the Icicle, eliminated this as a source of supply. A filtered supply from the Columbia

River, being cheaper and meeting the requirements of the State Board of Health, was adopted instead.

The Columbia River is the great natural drain for large parts of Washington, Oregon, and Idaho, and whenever it becomes necessary to use this water for a city supply, it must be put through a course of purification. The waters of many of the streams of this region have considerable colloidal clay in suspension, which should be considered in designing any filtration system. Mountain streams, although appearing to be clear, will almost always carry considerable silt. It is well known that rock-fill and earthen or timber dams gradually silt up and become tight with age.

In the main, information and experience gained in building and operating purification plants in the East will apply on the Pacific Coast, and with the information that is available, the engineer in the Northwest must familiarize himself with local conditions and then work out his own salvation.

The engineer will have some difficulty in convincing the people of the need for improvement and of the efficiency of his proposal. Many of them are "from Missouri" and have been hectored by men with something to sell. In 1913 the City of Centralia experienced a serious epidemic of typhoid which was traced to the water supply, which came from wells along the river. It was shown that seepage from the river was getting into the wells and was the source of the epidemic. Dr. Eugene Kelley, who was at that time Commissioner of Health of Washington, was asked by the city officials why he did not warn them of the danger impending. Dr. Kelley turned to the files of the local newspapers. In one of the glaring headlines it announced that Dr. Kelley had visited the city and, as a result of his investigations, was maligning Centralia by finding fault with the water supply and insisted that the dangers foreseen were purely imaginary. Had his warning been heeded this otherwise prosperous community would have escaped the sacrifice of the health of its people and an expenditure of a large amount of money. Needless to say, the City of Centralia now has a good water supply.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE SEWAGE DISPOSAL PROBLEM OF LOS ANGELES, CALIFORNIA*

By Willis T. Knowlton,† M. Am. Soc. C. E.

Synopsis

The continued and rapid growth of Los Angeles, Calif., has required constant attention to the matter of sewage disposal, and especially since 1920 when the population took such an upward trend that plans were required for a Metropolitan System. Studies were then made for the collection and disposal of sewage from all the area to the north and west of the city.

This paper describes how the outfall sewers required for this area have been planned to serve the Metropolitan Districts, regardless of political boundaries. Considerable data on the basis of design are given. The problem of ocean disposal of the effluent from the screening plants and the final disposal of the sewage screenings will require continued attention as the sewered population increases. In addition to sanitary sewage, the disposal of industrial waste is a problem to which reference is also made.

Early History

The City of Los Angeles commenced the construction of public sewers in 1873, when the population was about 7 000, although there are records of sewers as early as 1863. In 1887 the first comprehensive plan for a sewerage system was made by Frederick Eaton, M. Am. Soc. C. E., then City Surveyor. His plan, designed for sewage only, provided for intercepting sewers which were to discharge into an outfall sewer having a capacity for a total population of 200 000. This plan before its adoption was submitted to the late Rudolph Hering, M. Am. Soc. C. E., for his examination and approval. In 1900, the area of the city had increased 50% over its size in 1887, the total population was 102 479, and the system contained 160 miles of sewers.

Note.—Written discussion on this paper will be closed in December, 1927.

* Presented at the meeting of the Sanitary Engineering Division, Seattle, Wash., July 15, 1926.

† San. Engr., City of Los Angeles, Los Angeles, Calif.

The outfall sewer, as planned by Mr. Eaton, had a capacity of 30 sec-ft. As originally built, it contained two inverted siphons and its ocean end at Hyperion consisted of 24-in. cast-iron pipe laid into the ocean for a distance of 600 ft. This pipe was superseded in 1904 by a 30-in., steel pipe outfall which was laid on a pier to a distance of 940 ft. from shore. This steel pipe sewer was superseded in 1908 by a 34-in. wooden stave pipe, which had a capacity of 116 sec-ft., and provided for a population of 750 000. The wooden stave-pipe sewer discharged into a vertical pipe which carried the flow of sewage to a depth of about 6 ft. below mean sea level, where the water was about 16 ft. deep.

Growth of City

Owing to the continued growth of the city in all directions, the quantity of discharge into the main outfall sewer increased from an average daily flow of 15 000 000 gal. in 1910 to 23 000 000 gal. in 1915. This increase indicated that, to remedy complaints arising from this disposal, the sewage should be treated before discharge into the ocean at Hyperion. Accordingly, a bond issue for a sewage treatment plant at Hyperion was submitted to the voters in 1916, but was defeated. A similar proposition was submitted in 1917, but without success. On account of the World War, nothing was done until 1920, when the increasing volume of raw sewage discharged at this location made it necessary to plan for a new outfall as well as for the treatment of the sewage before disposal into the ocean.

In January, 1920, the population of Los Angeles was 576 673; in 1922, it was about 800 000, and a recent estimate (June, 1926) places it at 1 250 000. All estimates of future population must be considered as speculative, but a conservative estimate of the population of Los Angeles in 1950 would be not less than 4 000 000.

The sewage flow in the outfall during the past six years has increased in practically the same way as the population. In 1920, the sewage discharged at Hyperion averaged 33 000 000 gal. per day. In July, 1922, this flow had increased to 50 000 000 gal. per day, and, at present, the flow from all parts of the city, except the Harbor District, averages 78 000 000 gal. per day.

North Outfall Sewer Planned

Preliminary plans were made in 1920 for a new outfall sewer to supplement the existing one, and also for the treatment of the sewage. These plans provided not only for the conditions at that time, but also for a period of about thirty years thereafter. The plans contemplated the collection of sewage from all areas tributary either to the existing outfall or proposed new outfall, regardless of political boundary lines. The new outfall has been designated as the "North Outfall Sewer" and the existing outfall as the "Central Outfall Sewer".

After making plans for the North Outfall Sewer an educational compaign was required to acquaint the voters with the needs. Civic bodies and local

engineers approved the plans and recommended that bonds be voted for the work. Owing, however, to opposition from those who considered land disposal preferable to ocean disposal and who thought that there should be a reclamation of sewage for fertilizing values, a Special Sewage Disposal Commission was appointed to investigate the entire problem and submit a report. This Commission, consisting of George W. Fuller, the late George C. Whipple, and William Mulholland, Members, Am. Soc. C. E., confirmed the plans and recommended that the North Outfall be built for a capacity of at least 210 000 000 gal. per day and that the sewage be discharged into the ocean after passing through a fine screening plant. Opposition to the work proved, however, to be too great and the proposal was defeated in June, 1921.

Plans for another election were immediately commenced as the sanitary conditions in certain parts of the city were becoming intolerable. To obtain all necessary data, for acquainting the voters of the true situation, an examination of conditions in other cities was made by the City Engineer and the writer. The report of this examination, together with that of the Special Sewage Disposal Commission, was published in August, 1921, following which an educational campaign was conducted. As a result of these combined efforts, the election in August, 1922, approved the issue of sewer bonds.

Temporary Outfall Sewer

In 1922 a temporary relief sewer to carry the overflow from the Central Outfall was built as far as Ballona Creek, about 3½ miles below the point where the Central Outfall was intercepted by this relief sewer. Owing to complaints against the pollution of this creek, the City was ordered to take measures to prevent the discharge of raw sewage into the creek or provide for its treatment if the overflow was to continue. A coarse screening plant was built to remove all floating material before the flow in the Temporary Outfall was discharged into the creek. To prevent further pollution, the effluent from the plant was treated with liquid chlorine.

The upper end of the Temporary Outfall had a capacity of about 40 sec-ft., but owing to the increase of flow from new connections being constantly made to the system, the capacity of the lower end was increased to about 30 000 000 gal. per day. Records of the flow in this sewer from September, 1923, to May, 1924, gave an average of 28.2 lb. of chlorine and 13.4 cu. ft. of screenings per 1 000 000 gal. of sewage. The screens consisted of a double set of vertical units of ½-in. mesh arranged so that one set was in place while the other was being cleaned. The bottom of the channel of Ballona Creek was graded to assist in preventing a nuisance. However, the continued discharge of this treated sewage caused a second suit to be filed against the City by the City of Venice for conditions in the lower part of Ballona Creek where it empties into the ocean near Playa del Rey.

It thus became necessary to continue the construction of the Temporary Outfall to the ocean and to abandon the plans for the settling tanks and sprinkling filters to treat the flow in this outfall. A site of thirty-eight

acres had been purchased for this treatment plant, but opposition from property owners compelled the City to continue the Temporary Outfall to the ocean. This sewer having a length of 10 miles was used for disposal of the overflow from the Central Outfall into the ocean from April to December, 1924. The outlet end of the Temporary Outfall which was laid on the bottom of the ocean for a distance of 600 ft., from shore where the depth was 12 ft., consisted of 44-in. riveted steel pipe. The pipe was assembled on the beach, made air-tight and bulkheaded. It was then tugged into the water, floated into position, and sunk under a trestle that had previously been prepared for its protection. When the North Outfall was completed so that the Temporary Outfall could be abandoned, the material used for the latter was salvaged.

North Outfall Sewer Drainage Area

Immediately after the successful election held in August, 1922, contract plans were made for the construction of the North Outfall. It was decided at that time that this sewer should be divided into fourteen sections so that the work could proceed simultaneously at various points. Tunnel routes to Hyperion and along the brow of the Del Ray Hills were studied. Although longer, it was found that the Del Rey route would be cheaper, and accordingly it was adopted for the lower part of the North Outfall. The total length of this sewer, which was completed in November, 1924, is 17.6 miles. The discharge capacity at its lower end is 340 000 000 gal. per day.

When the outfall was proposed in 1920, it was planned to provide for the disposal of sewage from a large Metropolitan District, comprising all the area north of Manchester Avenue, then the southerly border of the city proper, and all area west of the easterly boundary of the city. This district included a large area in Los Angeles County south of Fernando Pass.

As it was necessary that that portion of Los Angeles County east of the city and south of Manchester Avenue should also plan for the disposal of its sewage, a tentative arrangement was outlined in November, 1924, whereby all areas tributary to the North and Central Outfall Sewers should be planned so to discharge their sewage. Similarly, those areas tributary to an outfall sewer, proposed to discharge into the ocean on the south coast of the County, would be accordingly planned. The dividing line between these two Metropolitan Areas was established by joint action of the engineers of the City and those representing the County Sanitation Districts. The area of the North Metropolitan District, which is tributary to the North Outfall Sewer, is about 513 sq. miles, and that of the South Metropolitan District about 605 sq. miles. On account of the size and continued growth of the city, there are sections which must have their sewage discharged into the so-called South Outfall Sewer which, when built, will be the main outlet for the South Metropolitan District.

In accord with the foregoing, the North Outfall was designed to carry the sanitary sewage from all parts of the city north of Manchester Avenue and include therewith the entire San Fernando Valley. The major part of the

Valley added by annexation about 170 sq. miles to the area of the city in 1915, and since then eight other annexations in the Valley have added about 24 sq. miles. There are, however, other cities and county areas in the North Metropolitan District which were considered in the design of the North Outfall. These cities include Glendale, Burbank, San Fernando, Beverly Hills, Culver City, Santa Monica, and El Segundo, which have a combined population of more than 140 000. In view of the rapid growth of this Metropolitan District and the demand for sanitary improvements, considerable study was required for future conditions.

Basic Data for Sewer Design

In the main part of the city it was thought that the business growth would cover the entire congested area extending from Figueroa Street to San Pedro Street and from Sunset Boulevard southerly to Washington Street.

Careful records were made of the water consumption in the different parts of the business district and a maximum allowance of 0.15 sec. ft. per acre and 0.02 sec-ft. per acre of floor area was provided. In the industrial districts a factor of 0.2 sec. ft. per acre was measured. Owing to the rapid increase in the number of laundries, creameries, canneries, and other plants discharging much waste water into the sewers, the use of zones created for such plants has proved to be of much benefit. In the Hollywood District it was found that water used for washing the films in the laboratories of the moving picture plants would add about 0.5 sec-ft. per plant. For an average of the industrial district it is believed that the waste water amounts at least to 10 gal. per capita per day.

Records of sewage flow in the residential districts show a variation of 30 to 80 gal. per capita per day. From gauging in several parts of the city it is believed that 60 gal. per capita represents the average flow, and the records of adjoining cities where sewer gauging has been made, give 50 to 70 gal. per capita for a similar flow. At a point on the Central Outfall Sewer, records of the sewage flow have been kept since 1906. These records show an increase in the average flow per capita during the period, 1922-26, of 72 to 82 gal., which is due to industrial growth with possibly some infiltration.

The problem of designing for the proper density for large areas in the Metropolitan Districts, which at this time are but partly developed, has received much study. The present population varies from 34 per acre in the center of the city to 10 per acre 5 miles from the center. For the entire municipal area the density is 4.2 persons per acre. In the San Fernando Valley an allowance of 1 person per acre was made in 1920, but in a recent study this factor has been increased to 3. It should be noted, however, that in the different centers of population in the San Fernando Valley, such as Van Nuys, Lankershim, and Owensmouth, the density to be planned for will vary from 15 to 30 persons per acre.

In connection with the study of the future population a check has been made of the present population by applying a factor to the number of regis-

tered voters, the school census, the number of water, gas, and electric meters in use, and the directory. For the registered voters the factor used has been 3.37; for the school census, 3.54; for water meters, 5.5; for gas meters, 4.0; for telephone service, 4.5; and for the directory census, 2.25. At present (July, 1926), the population of the City of Los Angeles is estimated at 1 250 000.

Hyperion Screening Plant

The construction of a screening plant at Hyperion was included as a part of the work of building the North Outfall Sewer. This plant is divided into two parts. In order to clarify the sewage from the Central Outfall at the earliest possible date and to prevent any further pollution of the adjoining beaches, the southerly plant was built and placed in service in 1924. The North Plant began operation in March, 1925. The combined capacity of the two plants is now 230 000 000 gal. per day, with space for additional units that will make a total capacity of 420 000 000 gal. per day.

These plants use drum type revolving screens and include pits, elevators, and pneumatic sludge ejectors with complete electrical equipment. At the South Plant there are eight 8 by 8-ft. screens, each with a capacity of 10 000 000 gal. per day. Each screen is driven by a 5-h.p. motor and has a separate enclosed bucket elevator driven by a 1-h.p. motor to raise the screenings into metal hoppers from which they are ejected by compressed air. The sewage entering the plant passes through inclined bar screens. At the North Plant only one-half the screens, or one row of five screens, is now in use. These screens are 14 ft. in diameter and 12 ft. long, each with a capacity of 30 000 000 gal. per day. Each screen is driven by a 40-h.p. motor and equipped with a screening elevator driven by a 2-h.p. motor. The screenings are discharged into metal hoppers 3 ft. in diameter and 10 ft. deep, from which they are ejected by air pressure. This plant is operated by six men in the day shift and two in each of the two night shifts. After leaving the drum screens, the effluent from each plant is measured by a Venturi recording meter.

Table 1 shows the volume of sewage flow that has passed through the Hyperion Plant, together with the quantity of screenings removed from the flow for an 18-month period.

The average cost of the maintenance and operation of the plant during 1925 was $105 per day, or $1.26 per 1 000 000 gal. At present, these screenings are buried in the sandhills adjacent to the plant, but studies are in progress to determine the feasibility of using screenings for fertilizing purposes after a sufficient reduction of the *B. coli* and moisture content.

Ocean Disposal

The screened effluent from the Hyperion Plant is discharged through the 7-ft. reinforced concrete pipe sewer laid below the bottom of the ocean for a distance of 5 093 ft. from shore, where the depth is 54 ft. below mean sea level. The outer end of this ocean section is divided into two 5-ft. branches which have an average length of 210 ft. from their junction with the 7-ft. sewer.

TABLE 1.—SEWAGE AND SCREENINGS AT HYPERION TREATMENT PLANT, JANUARY 1, 1925, TO JUNE 30, 1926.

Date.	TOTAL.		AVERAGE.		
	Sewage, in million gallons.	Screenings, in cubic feet.	Sewage per day, in million gallons.	Screenings, in cubic feet.	
				Per day.	Per million gallons.
1925:					
January	2 320	32 000	75	1 032	14
February	2 010	32 400	72	1 142	16
March	2 360	22 300	76	760	10
April	2 300	32 000	77	1 070	14
May	2 560	39 000	83	1 260	15
June	2 520	40 000	84	840	16
July	2 600	53 200	84	1 716	20
August	2 419	55 440	78	1 788	23
September	2 900	47 700	97	970	16
October	2 750	51 000	89	887	19
November	2 740	43 600	91	1 450	16
December	2 780	39 930	90	1 288	14
Total	30 259	488 170	83*	1 339*	16.1*
1926:					
January	2 412	49 880	78	1 609	21
February	2 402	43 300	86	1 544	18
March	2 450	49 130	78	1 585	20
April	2 569	50 710	86	1 690	20
May	2 514	53 030	81	1 705	21
June	2 302	50 190	85	1 726	20
Total	14 649	296 240	81*	1 643*	20*

* Average.

Biological surveys have been made since 1925 of the surf waters along the beach adjacent to the Hyperion Plant, to determine the number of *B. coli* present. These surveys made at semi-monthly intervals indicate that except for the mile directly in front of the plant and adjacent to the outfall there has been no count of more than 10 *B. coli* per cu. cm., which condition has been considered by the California State Board of Health as safe. In the outer ocean and on each side of the outfall, surveys were made in January, May, and October, 1925, and in January, 1926, to determine the area affected by the discharge of sewage. In all four surveys the sewage field containing 10 or more *B. coli* per cu. cm., has been outlined, and the results obtained are as given in Table 2.

On account of the discharge of raw sewage and the fact that there were some leaks in the outfall, the results obtained are not considered final and further surveys are being planned. However, it is considered that the discharge from the Hyperion Outfall receives sufficient dilution to prevent pollution of the adjacent beaches. After the repair of the leaks in the outfall, the effect of the diffusion and dilution should be much more in evidence than has previously been noted. The absence of any sewage odor, even over the point of discharge of the outfall, has been one direct result of the construction of the Hyperion Plant and Outfall Sewer.

In addition to the surveys for *B. coli*, the quantity of dissolved oxygen in the ocean waters adjacent to the end of the outfall has been determined at irregular times. These records indicate an average value of about 90% saturation within ½ mile radius of the sewer outlet, and at no point is it less than 84 per cent.

TABLE 2.—SURVEYS OF SEWAGE FIELD, LOS ANGELES, CALIF., FOR *B. coli* CONTENT.

Date.	Sewage field, in acres, containing 10 or more *B. coli* per cu. cm.	Approximate maximum sewage flow, in million gallons per day.	Acres per million gallons per day.	Acres per 1 000 population.
1925:				
January 21	700	115	6.1	0.70
May 11	305	115	2.6	0.30
October 1	360	120	3.0	0.36
1926:				
January 25	613	105	5.8	0.61

DISPOSAL OF INDUSTRIAL WASTES

Surveys are in progress to determine whether waste liquids and solids are being discharged into the sewerage system that should not be permitted to enter. Especial attention is given to the disposal of refuse from fruit and vegetable canneries. Such waste not only overtaxes and clogs the system, but causes additional cost for treatment at the ocean disposal plant. Gasoline and oil are not permitted to be discharged in the sewers, but such disposal is in evidence at times. Some consideration has been given in some cities to a special charge for the disposal of such industrial waste, but should not the remedy be made at the plant of the industry?

In this connection it may be added that in the disposal of sewage from other cities and county sanitation districts into the city system, a clause in the contract with the City of Los Angeles states that all requirements imposed by the City shall likewise apply to such outside areas, thus establishing a standard for the Metropolitan District. The City has made contracts for sewage disposal with seven outside cities and four County Sanitation Districts. The total sewage from these outside areas is estimated eventually to be 59 000 000 gal. per day. The total length of public sewers in the city on July 1, 1926, was 1 457 miles.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE WORK OF THE LOS ANGELES COUNTY (CALIFORNIA) SANITATION DISTRICTS*

BY ALBERT K. WARREN,† ASSOC. M. AM. SOC. C. E.

In May, 1923, the Governor of California approved an enactment known as the "County Sanitation District Act of 1923", providing for the creation, government, and maintenance of County Sanitation Districts. The governing body of these Districts is empowered to employ engineers and other persons; to acquire real or personal property and rights of way; to issue bonds of the district; and to levy and collect assessments on real property.

For several years the south coast cities of Los Angeles County—Manhattan Beach, Hermosa Beach, and Redondo Beach—had attempted to provide a satisfactory method of sewage disposal. Immediately after the Act became effective those cities united in what is known as the South Bay Cities Sanitation District.

Even before the Act became effective citizens of Watts, Compton, and the vicinity, took steps toward the formation of a district to include six incorporated municipalities and some contiguous unincorporated area. It was first believed that this area, known as County Sanitation District No. 1, was the only one having urgent need of sewage disposal facilities. It was contemplated that this District could resort to local disposal until other areas might desire to join in the construction of a more comprehensive system. To the contrary, however, other areas in the Metropolitan Area of Los Angeles County took action almost immediately, with the result that six districts had been formed by July, 1923, and since that time three additional districts have been organized.

There are now (1926) nine County Sanitation Districts in Los Angeles County, covering approximately 432 sq. miles and including 35 incorporated cities, 35 unincorporated communities, and a rather large rural area, with a total population of about 521 400, as shown in Table 1.

NOTE.—Written discussion on this paper will be closed in December, 1927.
* Presented before the Sanitary Engineering Division, Seattle, Wash., July 15, 1926.
† Chf. Engr., Los Angeles County Sanitation Dists., Los Angeles, Calif.

TABLE 1.—Sanitation Districts in Los Angeles County, California.

District.	Area, in square miles.	Present (1926) population.	Incorporated cities included within districts.
South Bay Cities....	11.89	20 000	Hermosa Beach, Redondo Beach and Manhattan Beach.
District No. 1.....	38.4	99 000	Huntington Park, Maywood, South Gate, Lynwood, Compton, parts of Vernon, Long Beach, and Los Angeles.
District No. 2.....	59.8	64 000	Montebello, and a part of Los Angeles.
District No. 3.....	51.7	121 000	Long Beach and Signal Hill.
District No. 4.....	2.2	12 000	Part of Los Angeles.
District No. 5.....	72.3	55 000	Inglewood, Hawthorne, Torrance, and a part of Los Angeles.
District No. 6.....	74.26	27 400	Whittier, and a part of Long Beach.
District No. 7.....	90	120 000	San Marino, South Pasadena, Alhambra, San Gabriel, Monterey Park, Sierra Madre, Monrovia, Arcadia, El Monte, Azusa, Glendora, Covina, and West Covina.
District No. 8.....	31.8	3 000	A part of Long Beach.

It was possible, therefore, to disregard any temporary expedient and to work for a final solution of the problem.

There is a well-defined drainage divide along the southern and eastern boundaries of Los Angeles. All the territory north and west of this divide drains naturally into the Los Angeles City Sewerage System which has already been constructed, and that south and east of the divide drains to the south, and with this the Los Angeles County Sanitation District plan is particularly concerned.

The Los Angeles City Sewerage System discharges through the recently constructed North Outfall Sewer. The South Bay Cities Sanitation District and County Sanitation District No. 4 lie within the area which drains to this system, and contracts have been made with the City of Los Angeles whereby the sewage from the two Districts will be discharged into the city system and disposed of through the North Outfall Sewer. Both these Districts have voted bonds to provide the necessary funds. Construction work in the South Bay Cities Sanitation District and County Sanitation District No. 4 is completed. County Sanitation Districts Nos. 1, 2, and 5 voted $9 120 000 in bonds in February, 1925, for the construction of the Los Angeles County Sanitation Districts Metropolitan Sewerage System.

In the plan of the Metropolitan Sewerage System there were five steps of procedure:

First.—The determination of a satisfactory plan for sewage disposal on which to base an estimate prior to a bond election.

Second.—The conduct of a bond election in each Sanitation District.

Third.—If the bond election is successful, to adapt the preliminary plan to one modified by conditions subsequent to the bond issue.

Fourth.—The construction of the project.

Fifth.—Its operation and maintenance.

County Sanitation Districts Nos. 1, 2, and 5, at present, are in the fourth stage. Districts Nos. 3, 6, 7, and 8, directly concerned in the joint undertaking, are prepared with a plan and are in a position to call elections to deter-

mine whether they will provide the necessary funds. Fig. 1 gives the outlines, physical characteristics, and plan of the Los Angeles County Sanitation Districts as organized.

In this paper, the writer will discuss the problems in bringing the formation and plan of the County Sanitation Districts to its present stage of completion, with particular reference to the engineering organization in determining a satisfactory means for sewage disposal prior to a bond election, and to the activities of the engineering organization in conducting the bond elections.

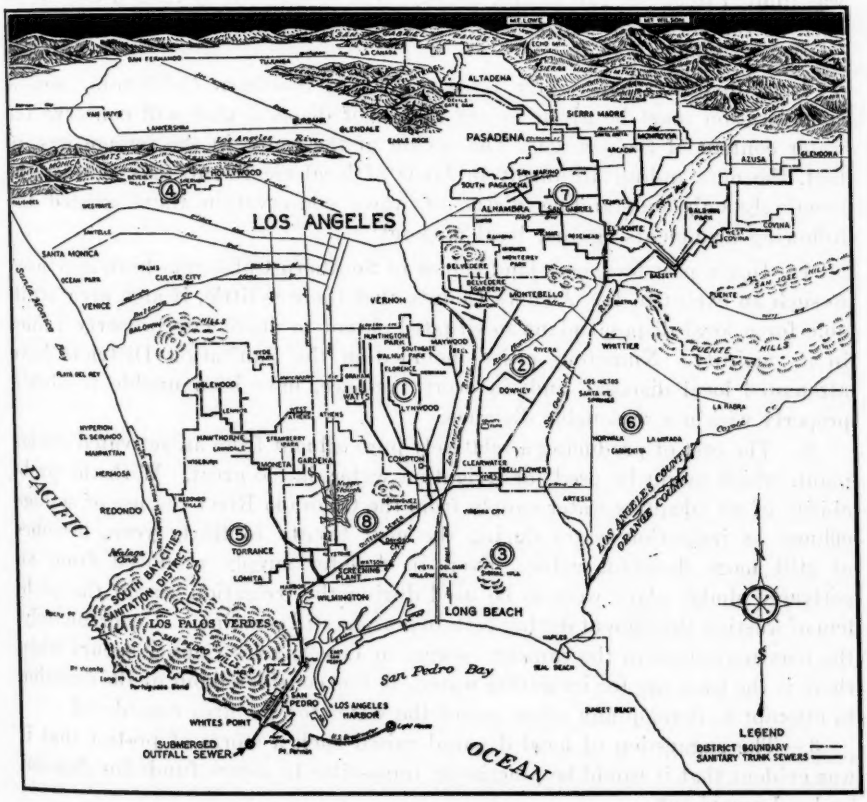


FIG. 1.—GENERAL MAP OF METROPOLITAN SEWERAGE SYSTEM IN LOS ANGELES COUNTY SANITATION DISTRICTS.

At the time the comprehensive sanitary sewerage system was planned for Los Angeles County, the North Outfall Sewer System of the City of Los Angeles, previously mentioned, was under construction. The North Outfall drained a well-defined area so that the problem was confined to an area lying south and east of Los Angeles and draining normally south to the ocean.

For such an area the natural process of accumulation would be through lines running in a general north and south direction accumulating sewage at, or near, the southerly ends of any districts formed. The final design for

the Sanitation Districts finds trunk sewers extending north and south through the Districts, converging with other trunk lines to the south and concentrating at the southerly boundaries of the several districts.

The Districts have steep slopes in the northerly extremities where the sewage flows are light, and relatively flat slopes in the southerly reaches where sufficient sewage will be concentrated to assure good velocities on relatively flat grades.

The design for a system for central accumulation was, therefore, comparatively easy, the principal problem being the means of disposal. The choice was limited to one of two means, namely, that of disposal by refined treatment, such as the activated sludge process; or by disposal in the ocean, sufficiently offshore to avoid nuisance.

In a district where water is scarce, as in Southern California, serious consideration must be given to any means of disposal that will conserve the water content of the sewage. The saving of the water in the sewage was, in fact, the outstanding argument in favor of local treatment. As against the seemingly only outstanding virtue of water conservation there existed the following arguments against local disposal:

1.—In the past few years land values in Southern California have increased to such an extent in Los Angeles County that there is little, if any, area available for a local disposal plant so situated as not to depreciate property values in its vicinity. Numerous cities included in the Sanitation Districts have attempted local disposal, and, in every instance, have been unable to obtain property within a reasonable distance.

2.—The cost of producing a relatively pure effluent from an activated sludge plant, which might be used for irrigation water, is too great. With the probability of an adequate water supply from the Colorado River the use of sewage effluent as irrigation water during the next twenty to thirty years becomes of still more doubtful value. Even if a water supply produced from an activated sludge plant were to be used during the irrigation season the problem of wasting the effluent during certain seasons still remains. Unfortunately, the maximum flow in the sanitary sewers in Los Angeles County occurs when there is the least use for irrigation water. It was considered out of the question to attempt to develop any other use of the water for the area considered.

3.—The suggestion of local disposal raised such a storm of protest that it was evident that it would be practically impossible to secure funds for disposal by local treatment.

4.—The activated sludge process cannot be considered as a permanent means of disposal. It is undergoing many changes in the method of application of air to sewage and the treatment and disposal of the sludge. Although many theories have been advanced for its solution, the method of sludge disposal still remains unsolved and, at best, presents a problem involving a tremendous expense in operation and construction.

5.—It was early determined that the cost of an activated sludge treatment plant was approximately the same as that of a joint outfall sewer to the ocean, if a site could be selected where such disposal would not create a nuisance.

As opposed to the activated sludge process, the Sanitation Districts had recourse to a plan for disposal in the ocean, provided a suitable point of disposal could be determined.

Economy of construction and of operation and maintenance, permanency, established practice, and custom, and deference to public opinion, all indicated that a jointly constructed outfall to the ocean would be the most adaptable solution.

The foregoing outlines briefly the problem presented to the County Sanitation Districts prior to bond election. The plan, as finally prepared, proposed a jointly constructed outfall to the ocean, and, based on this plan, the three Districts then concerned successfully carried bond elections by large majorities.

One of the most difficult problems was the control and regulation of a campaign prior to bond elections in the Districts. It was recognized that in a community with as many cities and communities involved, misinformation might spread rapidly, and that it would be much more difficult to correct a misapprehension than to register the correct information in the first place. For this reason it was decided that at every public meeting held in the interest of bond elections a representative from the Chief Engineer's office should attend to present facts. The Chief Engineer or his Principal Assistant, therefore, attended every public meeting where sewer bonds or sewage disposal was under discussion. The results were very gratifying. In each of the three Districts where bond elections were attempted under the plan as first proposed, the issues carried by overwhelming majorities.

There is only one other feature in the preparation of the plan for sewage disposal of the County Sanitation Districts that is of especial interest, namely, that of determining a future population on which to base the capacity life of the sewer system to be constructed. There is no well-established criterion for indicating a future population for districts in Los Angeles County. Few, if any, communities have ever made such forward strides in population as this area, and, therefore, predicted populations for Los Angeles County appear exceedingly optimistic if compared with past population increases for most counties of similar size in the United States.

In predicting future population, it was recognized that the population in certain areas would increase much faster than in others, and that no method of proportioning the increase directly over the entire district would be successful. The growth of each community, therefore, was studied separately, its future population predicted from year to year, and the future population curve of the district as a whole determined as a summation of the predictions for individual communities. Based on such a method the curves of growth for the districts are believed to be reasonably correct.

Prior to the design of the trunks sewers, a population density and concentration map was prepared, which served as a guide in predicting the quantity of sewage to be cared for at the expiration of a definite period of years. It was necessary to outline areas that would probably become industrial centers. It is impossible to predict the location and type of future industries, but

presuming a fair average of manufactured products, the quantity of waste from industrial areas has been determined with some degree of accuracy. No attempt was made to lump industrial wastes, but the total quantity to be cared for is a summation of the quantities resulting from predicted development of industries, based on predicted population growth.

In a project of this size, proposing to discharge a relatively large volume of sewage into the ocean, it is inevitable that the property owners near the outfall should feel uneasy regarding the outcome. For this reason the State Board of Health has deferred final action on a permit for the location of the ocean end of the outfall until a detailed investigation of the entire plan can be made. In the meantime, construction work is in progress on the interior trunks of the several Districts.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE LAKE WASHINGTON SHIP CANAL, WASHINGTON

By W. J. Barden* and A. W. Sargent,† Members, Am. Soc. C. E.

Synopsis

The Lake Washington Ship Canal, lying wholly within the limits of the City of Seattle, connects Puget Sound with a large fresh-water harbor comprising Salmon Bay, Lakes Union and Washington, and connecting waters, with a total area of about 25 000 acres. A dam and two locks, one large and one small, near the canal entrance hold the waters of the lakes 25 ft. above extreme low water of the Sound. This paper gives a general description of the locality, the original condition, a brief history of the improvement, and a description of the locks, dam, and appurtenances, and of the construction methods. The traffic developed is given, together with a summary of the beneficial results of the work. Under the headings, "Maintenance and Operation" and "Salinity", are given some of the results of the ten years' experience of operation and observation, with suggestions as to changes that might be desirable in future work of a similar character.

General Description

Seattle, the largest city of the State of Washington, is situated on Elliott Bay, an arm of Puget Sound, 47 miles south of the Strait of Juan de Fuca and 147 miles from the Pacific Ocean. Elliott Bay, 2 miles wide and 4 miles long, is protected from all severe storms and is accessible to the largest vessels afloat at all times. Deep water extends close to the shore except at Smith's Cove at the northern end and at the tide flats of the Duwamish River at the southern end. In these flats two waterways, each 750 ft. wide, 34 ft. deep, and 1 mile or more in length, have been excavated and the lower part

Note.—Written discussion on this paper will be closed in December, 1927.

* Col., Corps of Engrs., U. S. A., Seattle, Wash.
† Engr., U. S. Engr. Office, Government Locks, Seattle, Wash.

of the Duwamish has also been straightened and dredged to form a waterway suitable for commercial purposes with authorized depths of 15 to 30 ft.

To this commodious salt-water harbor the construction of the Lake Washington Ship Canal has added a fresh-water harbor of approximately 25 000 acres, comprising Salmon Bay, Lake Union, and Lake Washington, and connecting waters. (Fig. 1.) This improvement lies entirely within the city and extends from Puget Sound through Shilshole Bay, Salmon Bay, Lake Union, Portage Bay, and Union Bay, to deep water in Lake Washington. The distance by channel from deep water in Puget Sound to deep water in Lake Washington is approximately 8 miles. The tidal reach extends to the locks, approximately 1¼ miles. The extreme tidal range is 19 ft. The range between mean lower low water and mean higher high water is 11.3 ft. and between mean lower low water and extreme low water, 4 ft.

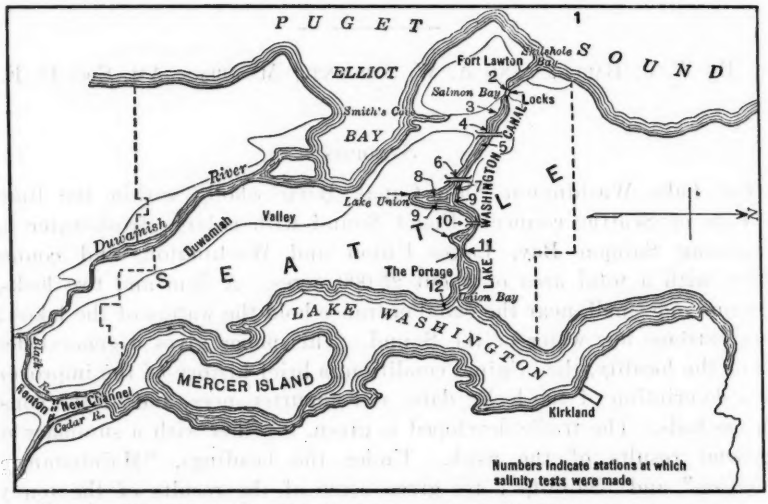


FIG. 1.—OUTLINE OF SEATTLE HARBOR AND LAKE WASHINGTON SHIP CANAL.

The double lock and fixed dam with movable crest are at the Narrows, at the westerly end of Salmon Bay, and hold the waters of Salmon Bay and the lakes 25 ft. above extreme low water in Puget Sound, with an allowable variation of 1 ft. above and 1 ft. below this plane.

The dredged channel below the locks is 300 ft. wide at the bottom and 34 ft. deep at mean lower low water, with a passing basin and log basin in the turn below the Great Northern Railway Bridge which is just below the locks and dam. (See Figs. 2 and 3.) From the locks to Lake Union the channel is 100 ft. wide on the bottom and 36 ft. deep. From the easterly end of Lake Union to deep water in Lake Washington the channel is 75 ft. wide at the bottom and 25 ft. deep. These dimensions will be increased to meet the needs of commerce. The main body of Lake Union required no dredging.

The large lock is 80 ft. wide and will accommodate vessels with a maximum length of 760 ft. and a draft of 36 ft. The small lock is 150 ft. long and 30 ft. wide, with a depth of 16 ft. over the sill.



FIG. 2.—LAKE WASHINGTON SHIP CANAL: LOCK AND DAM, LOOKING EAST FROM GREAT NORTHERN RAILWAY BRIDGE.

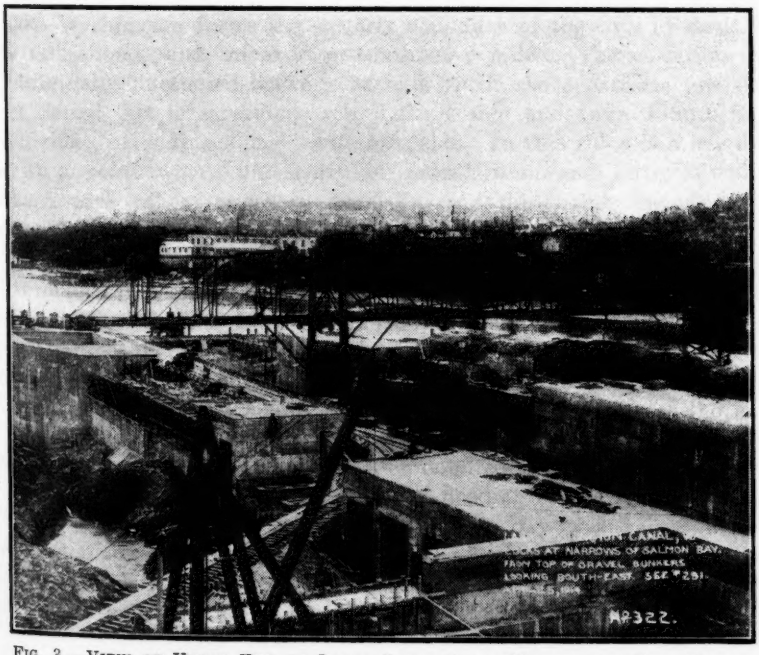


FIG. 3.—VIEW OF UPPER END OF LARGE LOCK, LAKE WASHINGTON SHIP CANAL, SHOWING RECESSES IN WHICH FOUNDATIONS FOR EMERGENCY DAM WERE CONSTRUCTED.





Original Condition

Originally, there was no connection between Lake Union and Lake Washington, and Lake Union had no navigable connection with Salmon Bay. Lake Washington, through the Black and Duwamish Rivers, had a navigable connection with Puget Sound with sufficient capacity for the transportation of logs.

Lake Union, the smaller of the two lakes, situated in the heart of Seattle, has an area of nearly 1 000 acres, of which 500 acres has a depth exceeding 40 ft. The general elevation of Lake Union was originally 25.5 ft. above extreme low tide in Puget Sound. Between Lake Union and Puget Sound is a high ridge, cut by a low valley at the extreme western end of the lake. Through this valley the natural drainage of the lake found its outlet to Salmon Bay.

Salmon Bay extends in a westerly direction to Shilshole Bay, an arm of Puget Sound. Salmon Bay and Shilshole Bay were originally navigable at high tide, but both were practically dry at extreme low tide. At mean lower low water a narrow, crooked channel, approximately 2½ miles long and 3 ft. deep, extended from Puget Sound to near the head of Salmon Bay.

South of this valley and of Salmon Bay are two high hills between which is a low narrow valley connecting the head of Salmon Bay with Smith's Cove, an indentation in the north end of Elliott Bay.

Lake Washington forms the easterly boundary of the City of Seattle. It is 19 miles long, with an average width of 2 miles. The elevation of the lake originally fluctuated between 33 and 37 ft. above extreme low tide in Puget Sound. It is separated from Lake Union and from Elliott Bay by a high ridge, extending along its western side. In this ridge is a low depression at a point where the shores of Lake Union and Lake Washington approach each other to within 2 000 ft. At this point, known as "The Portage", a narrow channel had been cut by private enterprise about 1888, for the passage of logs. The natural outlet of the lake was at the southern end through Black and Duwamish Rivers to the waters of Puget Sound at the southern end of Elliott Bay. Cedar River, which entered the Black River about ½ mile below the outlet of Lake Washington, is subject to annual floods, due to the melting of snow in the head-waters of the river in the Cascade Mountains. During the height of these floods the direction of the current in Black River between Lake Washington and the mouth of Cedar River was reversed, and a large quantity of water emptied into the lake, which acted as a storage reservoir preventing excessive flooding of the Duwamish Valley.

When the canal was constructed, Salmon Bay was raised and Lake Washington lowered to the level of Lake Union. Cedar River was diverted from the Black River into Lake Washington and the outlet through Black and Duwamish Rivers no longer exists.

History

The idea of a canal to connect Puget Sound with Lake Union and Lake Washington is practically as old as the City of Seattle, and the subject

received Federal recognition as early as December 9, 1867, in a report of a Board of Engineers for the Pacific Coast, which recommended the lakes as a site for a Naval Station on Puget Sound. From then until 1890 it was frequently referred to in official reports, but more from a naval than from a commercial point of view.

The River and Harbor Act of 1890 authorized the appointment of a board of three officers "to select and survey the most feasible location and estimate the expense of construction of a ship canal * * *". The Board's report, dated December 15, 1891, considered five possible routes. The present general route, being less expensive and having a better alignment, was considered preferable.

Several later investigations and reports were made, but the existing project, adopted by the River and Harbor Act of June 25, 1910, is based on the report of December 2, 1907, of the late H. M. Chittenden, M. Am. Soc. C. E. The terms of local co-operation required that, in addition to furnishing all rights of way, the County of King, or other local agency, should do the excavation in the waterway above the lock, and hold the United States free from any claim for damages on account of lowering the level of Lake Washington, raising the level of Salmon Bay, or any other alterations of the levels of any other part of the waterway.

In September, 1911, construction work was actually begun under the direction of the late Col. James B. Cavanaugh, Corps of Engineers, U. S. A., the District Engineer, who was in charge of the entire project, both as to Federal and County expenditures, until the completion of the main part of the work.

The total amount expended by the Federal Government has been approximately $3 500 000; King County has expended about $750 000, and the State of Washington about $250 000, in addition to the cost for right of way.

The cost of maintenance and operation, which is borne by the permanent indefinite appropriation for operation and care of canals, is about $80 000 per year.

Spillway Dam

The spillway dam (Fig. 4), extends from the small lock to the south shore of Salmon Bay. It is 240 ft. long, with six spillway openings, each 32 ft. long. A fish ladder for the passage of salmon to the spawning grounds above the locks, and an opening for discharging salt water from the upper pool, are provided through the dam. There are two walks extending the entire length of the dam, one at the elevation of the lock-wall, for pedestrians, and one at a higher elevation, from which the gates are raised and lowered. These are carried on arches spanning the piers which separate the spillway openings.

The fixed part of the dam is of ogee section with a vertical up-stream face and a toe 4 ft. long extending up stream. The total width of the base is 46 ft. A concrete apron, with its top 10 ft. below extreme low tide, extends 114 ft. below the dam. The total height of the dam, not including cut-off walls, from the foundation to the spillway crest, is 37.75 ft. The latter is

17.75 ft. above extreme low tide. The six spillway openings are closed by Taintor gates. The gates are arranged to rotate around the trunnions until they rest on the lower footwalk of the dam, when necessary for painting or repairs. The gates are raised and lowered by two chains which are attached when desired to cross-heads on an electrically driven machine which can be moved from gate to gate on the upper walk of the dam. The cross-heads are moved back and forth by means of revolving screws, and a gate can be raised wide open in one operation.

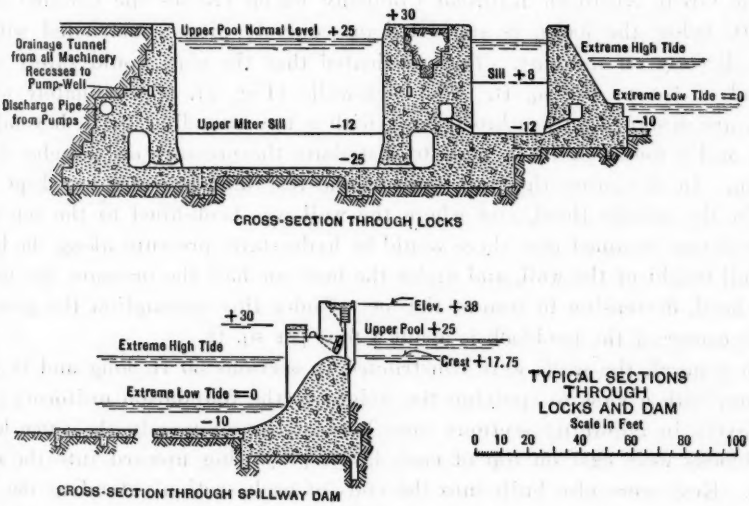


FIG. 4.

The capacity of each spillway opening is 2 350 sec-ft. at normal upper pool level, and with all the gates open no trouble with scour has been experienced beyond the lower edge of the apron. The dam was constructed in sections, 36 ft. long, each section consisting of three horizontal layers well keyed together. After two horizontal layers had been placed the piers were constructed, then the spillway sections, after which the footwalks were built.

LOCKS

Table 1 gives the principal features of the locks.

TABLE 1.—PRINCIPAL FEATURES OF THE LOCKS, LAKE WASHINGTON SHIP CANAL.

Item.	Large lock.	Small lock.
Clear width of chamber, in feet	80	30
Maximum available length, in feet	760	123
Maximum lift, in feet	26	26
Depth on upper miter sill at low water in upper pool, in feet	36	16
Depth on intermediate miter-sill at mean lower low water, in feet	29	....
Depth on lower miter-sill at mean lower low water, in feet	29	16

The large lock is 825 ft. long and is divided into two chambers by an intermediate gate, the lower chamber being 375 ft. and the upper one, 450 ft. long. The small lock, alongside and south of the large lock, is designed to accommodate with the minimum of delay the large number of launches, tugboats, and small craft continually passing in and out of the canal.

The locks are constructed on a bed of hard impervious clay. At the upper end this clay bed slopes off rapidly, but below the locks the hard formation extends almost to Puget Sound. The double-track bascule railway bridge of the Great Northern Railroad Company which crosses the channel about 700 ft. below the locks, is supported on concrete piers constructed without piles directly on the clay. Tests indicated that the clay would safely carry a load of 5 tons per sq. ft. The lock-walls (Fig. 2), approximately 62 ft. high, are designed with a broad base, with a toe extending 10 ft. beyond the face, and a toe-block 10 ft. wide to distribute the pressure on the clay foundation. In designing the wall sections the resultant pressure was kept well within the middle third, and where the wall was back-filled to the top with earth it was assumed that there would be hydrostatic pressure along the back, the full height of the wall, and under the heel one-half the pressure due to the total head, decreasing to zero at the toe. Under this assumption the pressure at the center of the toe-block is about 4 tons per sq. ft.

In general, the walls were constructed in sections 30 ft. long and in 5-ft. courses, with a view to applying the weight to the foundation uniformly; the top layers in adjoining sections were kept at approximately the same level. Key-blocks were cast on top of each layer, projecting upward into the next layer. Keys were also built into the ends of each section extending the full height of the wall. Concrete cut-off walls, 4 ft. deep, or more, and 18 in. thick, were built under the heels of the walls, under the miter-sills, and under the entire length of the dam to the south bank. Below the floor level, lead stop-waters were placed between all wall sections, down the cut-off walls, between floor sections, and between the toe of the wall and the floor; also, between the sections of the dam. At the upper end of the large lock beyond the guard-gate, where the foundations were not as satisfactory as over the remainder of the area, cellular walls were built.

The floor of the large lock, which is only 3 ft. thick, was designed merely as a pavement to prevent scour in the lock. No relief valves were provided against hydrostatic pressure, reliance being placed on the cut-off walls to prevent excessive leakage. No difficulty has been experienced from this source when the lock is unwatered; in fact, holes drilled through the floor a few years ago to ascertain if there was any pressure, showed that, although a small quantity of water had accumulated under the floor, no pressure had developed due to the 50-ft. head along the outside of the lock, showing that the precautions taken as to cut-off walls and lead stop-waters had been successful.

Gates.—The gates of both locks are of the mitering type. In the large lock there are five gates, the upper and lower guard-gates, and the upper, intermediate, and lower service gates.

The lower guard-gate, which is used only when the lock is unwatered for purposes of repair and inspection, is designed to withstand a head of 43 ft. It is 46 ft. high and each leaf weighs 207 tons. The lower service and intermediate gates are each 56 ft. high and designed to withstand a head of 26 ft. Each leaf weighs 237 tons. The upper service and guard-gates are smaller, being only 43 ft. high, but are subjected to a head of 38 ft. when the lock is unwatered. Each leaf weighs 180 tons.

All gates of the large lock are constructed with horizontal girders spaced equi-distant on 3½-ft. centers, and five transverse vertical frames built in between the girders extending from the bottom to the top of the leaf. The skin-plates extend the full height of both up and down-stream sides of the gate. The lower part of the gate forms an air chamber to lighten the weight on the pintle and anchors, while water is allowed to run freely in and out of the upper part, through openings in the up-stream skin just above the air chamber and through the girders and vertical frames.

Provision is made for blowing out by compressed air any water that may enter the air chamber. A 2½-in. pipe extends down to and is attached to the top of the chamber. Through this pipe a 1¼-in. pipe extends almost to the bottom of the gate, the air entering through the large pipe and the water being forced out through the smaller one.

Access is provided to all parts of the interior of the gates through openings in the girders and vertical frames and through the manhole which extends from the top of the gate, through the water chamber, into the air chamber.

The wall reaction bearing consists of a structural steel, built-up member embedded in the concrete, to which are bolted steel castings which have a slightly concave surface, with which the convex reaction casting on the gate comes in contact when closed. The miter edge of the gate has adjustable steel bearing pieces extending the full height, behind which Babbitt metal was poured to bring it into proper alignment.

The forged steel pintle is supported on a steel casting embedded in the concrete. The top surface is hemi-spherical in shape, and the bushings on which the gates rotate are of vanadium steel held in place in a steel casting on the heel of the gate. The pintle is free to move on its foundation whenever the gate is forced out by obstructions on the sill.

The gate anchors are heavy steel trusses embedded in the concrete and connected with the gate by forged steel eye-bars and pins. The eye-bars are bronze-bushed and are connected by sleeve nuts with a square thread, 1-in. pitch in the one half and ¾-in. pitch in the other half, both right-hand threads, so that when the coarse thread is being taken up the finer thread is being unscrewed, and *vice versa*.

At the lower edge of the down-stream side of the gate greenheart timbers were fastened into a structural steel channel. When the gate is closed, the timber comes in contact with a similar timber fastened in a steel chair on the miter-sill. After four years of service it was found that the timbers on the lower guard-gate of the large lock had been honeycombed by marine borers and that it would be necessary to replace them with other material. Iron

bark was tried, but in two years it was found that this also had been destroyed by marine borers and that the timbers attached to the miter-sill were partly destroyed. Considerable study was then given to determine the best method of overcoming the difficulty. It was thought impracticable to replace the timbers on the miter-sill by diver. A coffer-dam could only be constructed at considerable cost and would also necessitate the closing of the lock for a considerable period. The plan finally adopted was to remove the timber on the sill with the assistance of the diver. The gate was then removed and docked, and steel castings were made and attached to it. These castings (Fig. 5) extend out so as to come in approximate contact with a steel chair which formerly held the timbers on the miter-sill, and take the pressure in case of failure of the leaves to miter properly. A lip extending downward from the castings carries a heavy piece of rubber belting which forms the water-seal between the gate and the concrete sill. This arrangement proved to be satisfactory and a similar device has been placed on the lower guard-gate of the small lock. No such trouble has been experienced with any of the other gates, probably because the service gates are regularly used and the water at the upper guard-gates is mainly fresh.

The gates of the small lock (four in number, two service and two guard) are composed of horizontal I-beams with skin-plates on one side only. Mitering devices, consisting of a jaw at the top of one leaf, which engages a post on the other leaf as they swing together, are provided on all gates.

Culverts and Valves.—The culverts in both locks are in the side walls, with laterals extending into the chambers at the floor level (Fig. 2). The ends of the culverts are between the service gates and the guard-gates, and all gates and valves are accessible when the lock is unwatered, except the exterior surfaces of the guard-gates.

The culverts of the large lock are 8½ ft. wide and 14 ft. high, with corners rounded to a radius of 3.5 ft. The intake end is increased in size to 16 ft. in width and 14 ft. in height, and the discharge ends are 10 ft. wide and 16 ft. high. Both ends turn at an angle of 90° horizontally into the lock. The bottoms of the culverts are at the floor level except at the discharge end where they are raised 5 ft. with a view to overcoming difficulties attendant on the difference in density of the water above and below the gates. The laterals are 4 ft. wide and 2 ft. high. There are twenty-four entering the upper chamber and twenty-eight for the lower chamber.

The valves of the large lock are of the Stoney gate type, raised and lowered by means of two vertical screws revolving in fixed nuts in the cross-head which is attached to the valve stem. The valve stem passes through a stuffing-box in a water-tight bulkhead which closes the bottom of the machinery recess. The latter is about 22 ft. below high-water level in the lock. The valves are equipped with roller trains which are raised and lowered with the valves, but at half speed, by pulleys over which a chain is taken. One end of the chain is fastened to the cross-head on the valve stem and the other passes through a stuffing-box in the water-tight bulkhead over the valve. The valves of the small lock are of the cylindrical type.

Signal lights installed in the controllers in the operating houses indicate whether the valves are in the open or closed position. Limit switches are provided for controlling the travel of the valves.

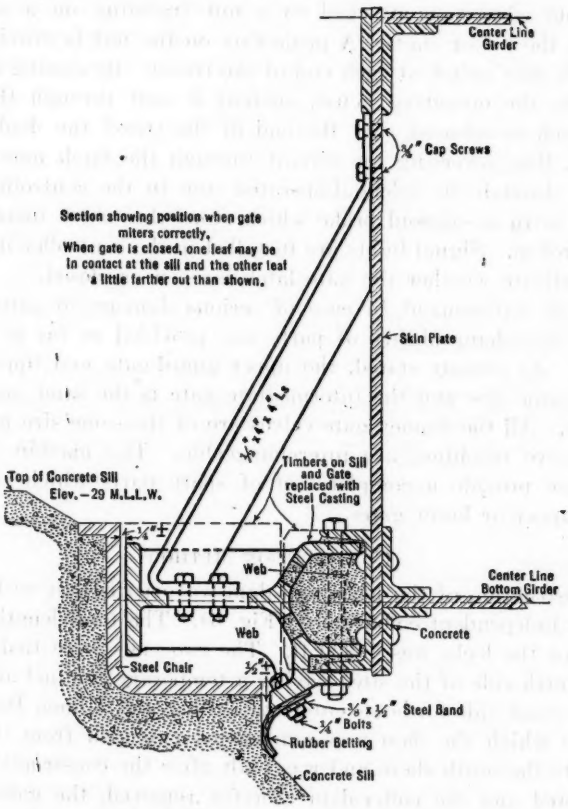


FIG. 5.—LAKE WASHINGTON SHIP CANAL, WATER SEAL, LOWER GUARD GATE—LARGE LOCK.

OPERATING MACHINERY

The gates are operated by electrically driven winding drums acting through cables attached to the gates near the outer end of each leaf. The cables pass from the drum over rollers and round sheaves, through passages in the lock-walls to the two sides of each gate leaf, the operation of each leaf being entirely independent of the other. The opening cable is 1 in. in diameter and the closing cable ⅞-in., 6/19 plow steel rope. The drums are spirally grooved, driven in the large lock by 20-h.p. motors through worm and spur gears. The drums are connected with the spur gears through adjustable slip friction clutches arranged so that in case of an excess load the clutch will slip. The radii of the drum spirals are proportioned so that there will be no excess slack in the cable as the gate swings through its arc of revolution. The ends of the cable are attached to the drums through adjustable screw fastenings, in order that the slack due to the stretch of the cables can be taken up.

Gate latches are provided for each leaf of the service gates of the large lock to hold them securely in the open position. A movable latch bolted to the lock-wall engages a catch on the gate leaf. The latch is operated by a flexible wire rope attached to a nut traveling on a screw which is attached to the motor shaft. A projection on the nut is provided for reversing a double pole switch at each end of the travel. By closing a switch in the controller in the operating house, current is sent through the latch motor and the catch is released. At the end of the travel the double pole switch is reversed, thus reversing the circuit through the latch motor, and closing the circuit through the solenoid-operated stop in the controller. The motor is provided with a solenoid brake which stops the motor instantly when the circuit is broken. Signal lights are installed in the controller in the operating house to indicate whether the gate latch is open or closed.

For quick replacement in case of serious damage to gates or operating machinery interchangeability of parts was provided as far as practicable in the design. As already stated, the upper guard-gate and upper service gate are of the same size and the intermediate gate is the same size as the lower service gate. All the Stoney gate-valves are of the same size and all parts of gate and valve machines are interchangeable. The machines at the intermediate gate provide a complete set of spare parts which can be used at either the upper or lower gates.

Construction Methods

The construction of the locks and dam was carried out within the protection of two independent coffer-dams (Fig. 6). The total length of the coffer-dam wall for the locks was 1 320 ft. The two ends were tied into the bank along the north side of the site, leaving a temporary channel along the coffer-dam on the south side for boats entering and leaving Salmon Bay. The coffer-dam within which the dam was constructed extended from the wall of the small lock to the south shore and was built after the construction of the locks was completed and the coffer-dam therefor removed, the gates of the large lock being left open for navigation.

The coffer-dam walls, which were 20 ft. apart, were constructed of round piles, wales, and Wakefield sheet-piling, and were tied together by iron rods. The sheet-piles were driven a short distance into the underlying hard material, not quite to refusal. It was found that if they were driven too hard it was difficult to hold them up close together, and they had a tendency to move off line away from the wales. A better job was obtained by not driving them to refusal. The material for filling and banking was supplied from excavation in the lock-pit. This material was excavated by dipper dredges before the coffer-dam was completed, and the foundation was cleaned to grade by pick and shovel. The surplus material was removed by crane and skip to the bank just inside the coffer-dam. Although at high tide the deepest part of the excavation was 58 ft. below water level, there was practically no seepage through the coffer-dam or through the clay foundation into the lock-pit, and two or three 2-in. centrifugal pumps, at different points within the enclosure, were more than ample to handle all seepage and rain water.

All concrete work in the locks and dam, the construction of permanent buildings, and the work on the grounds were done by hired labor and Government plant. Separate contracts were let for the excavation and coffer-dam, gates, machinery, permanent pumping plant, and the removal of the coffer-dams, as well as a number of smaller contracts for minor parts of the work.

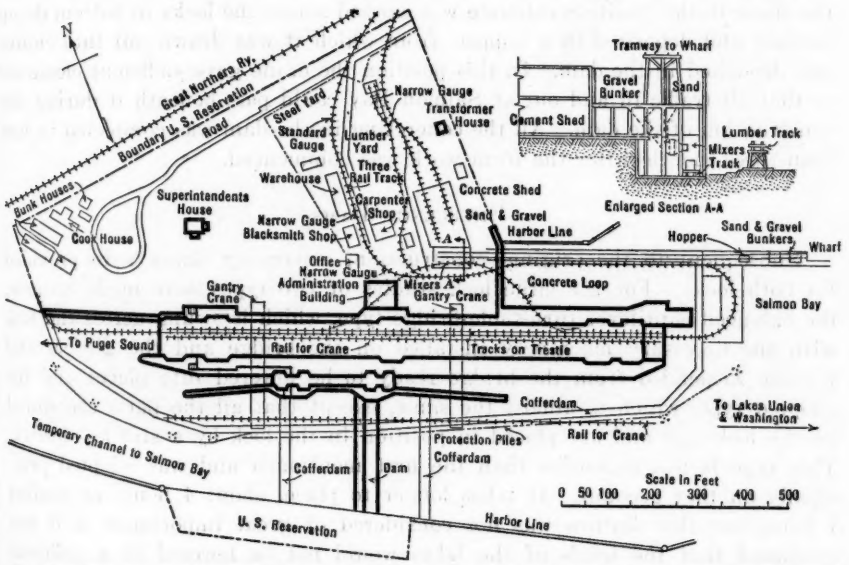


FIG. 6.—GENERAL LAYOUT, WASHINGTON SHIP CANAL, PERMANENT LOCKS AND DAM (HEAVY) AND CONSTRUCTION PLANT (LIGHT).

All concrete in the locks was handled by two gantry cantilever cranes which were constructed across the lock-pit, one rail being along the axis of the lock just above the floor level and the other partly on top of the coffer-dam and partly on pile trestle. Each crane was equipped with two 30-h. p. electric hoists which operated two trolleys, each capable of lifting 5 tons. The concrete was delivered from the mixing plant, consisting of three 1½-cu. yd. mixers, into bottom dump buckets, which were carried on cars by steam locomotive to the cranes, where they were picked up, two at a time. Six buckets were placed on the cars in each train so that two could be filled simultaneously at the mixers, and two picked up at the same time by the crane. As much as 1 140 cu. yd. was placed in an 8-hour day, and the highest monthly average was 24 000 cu. yd. Approximately, 150 000 cu. yd. was placed in 9 months (from May, 1913, to January, 1914). The bulk of the concrete in the locks was placed at a cost of $4.00 per cu. yd., and the average cost of all concrete in the locks, dam, and permanent buildings was approximately $5.50 per cu. yd.

The forms were mainly of the cantilever type, fastened to the wall with ⅞-in. bolts, 2 ft. long, with threads on both ends. The outer end was made square so that it could be easily turned by a wrench while the concrete was setting. On the inner end a nut and a washer 3 in. square were used, which

were left in the concrete. The bolts were removed when the forms were taken off and were used again repeatedly. Each form was used five or six times and then knocked down, the lumber being utilized for building rough forms.

After the completion of the locks one of the gantry cranes was lifted up and supported on top of the lock-wall, and on a tower near the dam. With the crane in this position concrete was carried across the locks in bottom dump buckets and deposited in a hopper, from which it was drawn out into chutes and deposited in the dam. In this position the crane gave sufficient clearance so that all traffic in and out of Salmon Bay could pass beneath it during the construction of the dam. All the concreting in the dam was completed in less than three months after the form work was commenced.

Emergency Dams

Subsequent to the original construction, emergency dams were provided for both locks. For the large lock studies of two types were made, namely, the balanced cantilever turn-table bridge type, which is swung across the lock with the necessary machinery installed on the bridge and the girders and wickets suspended from the bridge ready to be lowered into place; and the adopted type, which is almost the same, except that all the parts are stored on the lock-wall and are placed in position in the lock by a stiff-leg derrick. This type is less expensive than the first mentioned and was adopted principally on that account. It takes longer to place, about 4 hours as against 1 hour, but this feature was not considered of great importance as it was estimated that the levels of the lakes would not be lowered to a sufficient degree during the time required for placing it to cause serious damage.

Figs. 7 and 8 show the dam in storage position on the lock-wall and a down-stream view when in position. Fig. 9 shows a cross-sectional view. The dam consists of two removable bridges which span the lock when in use, six wicket girders which form the framework, twenty-four wickets which complete the closure, a stiff-leg derrick, and the necessary operating machinery for handling the various parts. Needles are also provided for closing the clearance between the vertical rows of wickets.

The wicket girder bridge consists of a horizontal truss which supports the upper edge of the dam and two vertical trusses which support the dead load. It is placed by the derrick in recesses provided in the lock-walls. Its weight is approximately 72 tons.

The operating bridge is placed about 35 ft. up stream from the girder bridge and carries the machinery for lowering the wicket girders against the sill in the lock floor which supports the lower edge of the dam. The machinery consists of a two-speed electric hoist with six drum units operated through a line shaft, each unit consisting of two drums which are geared to the line shaft and are engaged or disengaged by means of jaw clutches. Each drum unit operates the hoisting tackle which is provided for lowering a wicket girder.

The six wicket girders, the skeleton of the dam, are delivered one at a time by the derrick in a horizontal position. They are then lowered until the

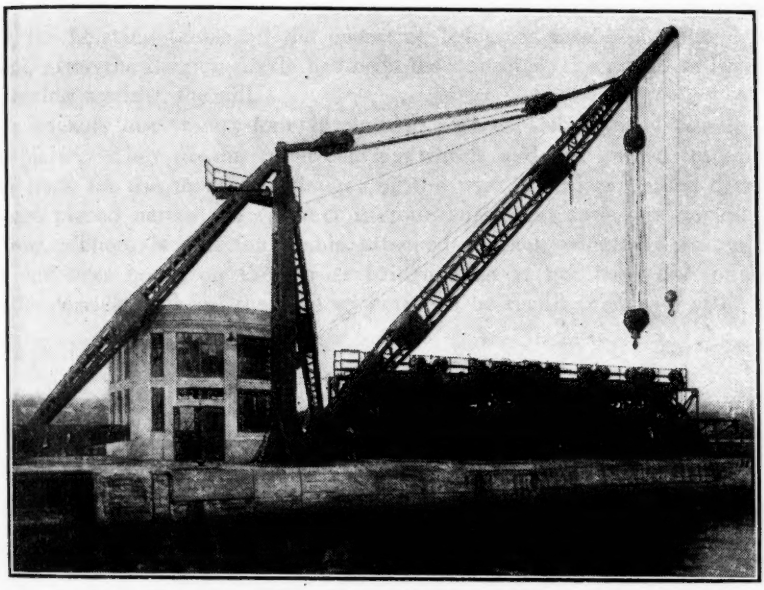


FIG. 7.—LAKE WASHINGTON SHIP CANAL: EMERGENCY DAM IN STORAGE POSITION ON LOCK WALL.

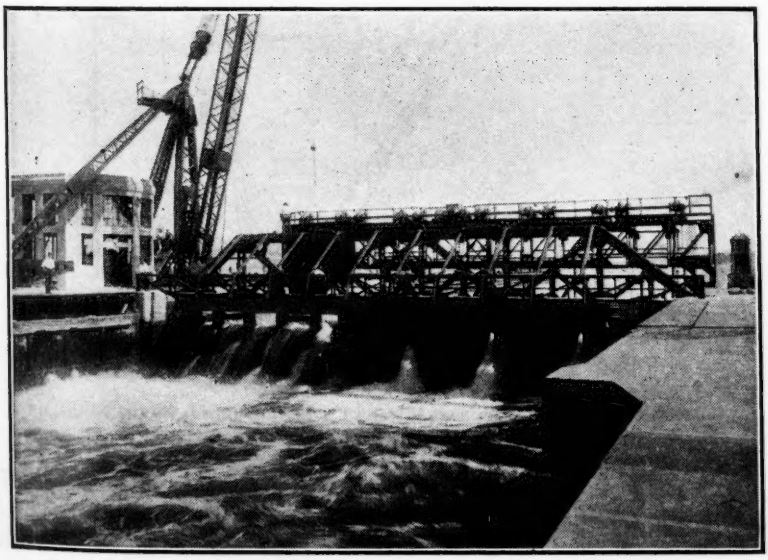


FIG. 8.—VIEW OF PLACING NEXT TO LAST WICKET, LOOKING UP STREAM, EMERGENCY DAM, LAKE WASHINGTON SHIP CANAL.





upper end is hooked over pins provided on the up-stream face of the girder bridge, the hoisting tackle on the operating bridge is attached to the lower end and, after the derrick tackle has been disconnected, the girder is lowered to a bearing against the sill.

The wickets are twenty-four in number and consist of steel frame and buckle-plates. They are on roller-bearing wheels and are guided into place over a track on the up-stream flanges of the wicket girders by the derrick. They are placed across the channel in four horizontal rows, six wickets in each row. There is sufficient cable attached to each wicket to fasten the upper end over hooks on the girder bridge after it has been disconnected from the derrick hook, so that the wickets can be readily removed after use.

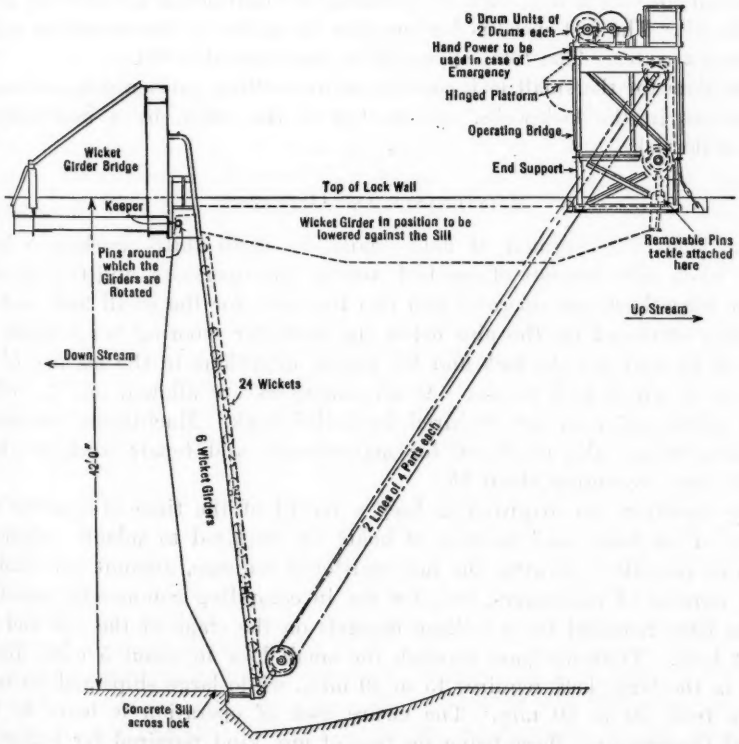


FIG. 9.—LAKE WASHINGTON SHIP CANAL EMERGENCY DAM: CROSS-SECTION OF DAM IN THE LOCK.

The capacity of the derrick is 75 tons at 60-ft. radius. The mast is 45 ft. high and the boom 102 ft. long. The main hoist tackle has twelve parts and is used only for handling the wicket girder bridge and the operating bridge. Farther out on the boom is an auxiliary tackle designed for the rapid handling of the wicket girders and the wickets. The wickets are hoisted on a single line at 125 ft. per min., and the girders are handled at half this speed by hooking the end of the line to the boom and hanging a snatch-block in the

bight. The boom tackle is designed for a load of 309 000 lb. and has seventeen parts. The maximum vertical uplift on the stiff legs is 300 000 lb.; the speed of the boom fall is $93\frac{7}{10}$ ft. per min.; and that of the main hoist and auxiliary hoist falls is 125 ft., per min. The speed of the swinger cable is 31 ft. per min.

The derrick is operated by an electric hoist with three drums, each drum being connected through worm gearing to its own 100-h.p. motor. The swinger hoist is of standard design operated by a 40-h.p. motor. The hoist on the operating bridge for lowering the wicket girders in the current is operated through a worm by a 40-h.p. motor. It has two speeds, 8 and 32 ft. per min., respectively. The slow speed is for raising the wicket girders in the current in case of an emergency, such as releasing an obstruction between the girder and the sill. The fast speed is for lowering the girder in the current or raising it in quiet water. The total approximate cost was $175 000.

The dam for the small lock consists of five rolling gates which are lowered in recesses in the lock-walls, one on top of the other, by a hand-operated stiff-leg derrick.

Maintenance and Operation

The locks are operated 24 hours daily, by three shifts working 8 hours each. Each shift consists of one lock master, two operators, and two linesmen for the large lock, one operator and two linesmen for the small lock, and one pier man stationed on the pier below the locks for mooring boats which are required to wait for the lock and for giving directions to the masters of the boats as to which lock to use. As all employees are allowed one day off in seven, additional men are required for relief work. Machinists, carpenters, and laborers are also employed for maintenance and repair work, the total regular force averaging about 45.

The operators are required to keep a record of the time of entering and leaving of all boats, and masters of boats are required to submit statements on forms provided, showing the net registered tonnage, amount and kind of cargo, number of passengers, etc., for use in compiling commercial statistics.

The time required for a lockage depends on the stage of the tide and the size of boat. Tugboats pass through the small lock in about 5 min. Barge traffic in the large lock requires 15 to 20 min., while large ships and log rafts require from 30 to 50 min. The entire cost of operation is borne by the Federal Government, there being no fees of any kind required for lockage.

As was to be expected, some difficulties have been experienced with the opening of the gates and in navigation through the locks due to the difference in density of the fresh water above and the salt water below. In theory, this difference would cause the flow through the culverts in filling or emptying the lock to cease when equal pressures were reached there, which would be before the level above the gate had fallen to that below, and there would remain a positive pressure against the lock-gates making it difficult to open them. In practice, this is so in different degrees dependent on the stage of the tide, the admixture of salt and fresh water, the somewhat different condi-

tions at the upper, intermediate, and lower service gates, and the methods of lockage.*

At the upper gate of the locks, which is generally used as such, and with either the intermediate or lower gate used as a lower gate, this difference in pressure is quite marked. The difference in head is usually 3 or 4 in., but sometimes it is as high as 6 in. There is some over-travel of the water through the culverts with temporary corresponding rise of the water in the lock, as explained subsequently, and advantage is taken of the resulting diminution of the pressure against the gates to open them before the condition of equilibrium has been reached. When the difference in head is more than 4 in., it is preferable in order to reduce the strain on the cables and operating machinery to freshen the water in the lock by lowering and refilling before the gate is opened.

When the lower chamber is used and the intermediate gate becomes the upper gate there is a marked over-travel of the water in the culverts before the filling ceases and at the same time the forebay is temporarily lowered so that the water in the lock rises slightly higher than the level above the gates which thus become subjected to a reverse head tending to open them. This condition lasts only momentarily, and equilibrium is soon reached, with the level of the water above the gates higher than that below, due to the difference in density. As an illustration the following example is given: With the upper pool at Elevation 25 ft. 7 in. and the tidal elevation at 8 ft., the lower gate was opened and time allowed for the water in the lock to reach full salinity. The intermediate gate was then closed, the upper chamber filled, the upper gate opened and time allowed for the water to become quiet. The lower gate was then closed, the valves of the intermediate gate opened, and the lower chamber filled. The water in the forebay, that is, the upper lock chamber, was lowered somewhat when the valves were first opened and surged up and down for a few minutes. The lower chamber filled to Elevation 25 ft. 7 in. The miter broke and actually pushed the gate away from the sill until held by the cables. No attempt was made to open the gate and in less than a minute it moved back against the sill. The water in the lower chamber lowered quite rapidly 7 in. to Elevation 25 ft., while above the gate the water became quiet at Elevation 25 ft. 7 in., or the upper pool level. The pools then remained with this difference of elevation of 7 in. against which it was impracticable to open the gate. To do this it was necessary to lower the water in the chamber and re-fill. Under substantially similar conditions the difference in head after equilibrium has been established, when the upper gate is used, is about 5 in. The over-travel of the water and the surge in the pool elevations is also less, probably partly due to the shorter length of the column of water in motion through the culverts. Due to these more unfavorable conditions at the intermediate gate the lower chamber is seldom used, the smaller boats being always taken into the upper chamber when only part of the lock is required.

* Quite a complete mathematical discussion of this matter will be found on pp. 85–99 of the Annual Report of the Isthmian Canal Commission for 1911, together with measures proposed to obviate the anticipated difficulties that would arise therefrom.

Not much difficulty is experienced in the opening of the lower gate or of the intermediate gate when used as a lower gate. The miter frequently breaks, probably due to the over-travel of the water through the culverts and the momentary surge in elevation above and below the gate. After equilibrium is reached there is some difference in elevation and some head on the gate, but generally it can be opened easily. There does not appear to be much difference in the conditions at the two gates.

Immediately after opening the lower gate a strong surface current sets up, which lasts for several minutes, running out of the lock with the corresponding under-current running in. This is troublesome to the smaller vessels entering and leaving the lock, and they are often held in the lock for several minutes until the current subsides. Vessels with tows are also delayed several minutes when leaving the lock at the upper end due to the strong surface current flowing into the lock when the upper gate is opened and the salt water in the lock chamber flows up stream.

The locks are unwatered at least once each year for painting the service gates and for general repairs to machinery, valves, etc. The lower guard-gate of the large lock, which is in salt water, is removed and painted at least once every 4 years and the upper guard-gate once in about 6 or 7 years. This is accomplished with the assistance of the snagboat *Swinomish*. The openings for admitting water into the water chambers are closed at low tide and the water remaining in the chamber is pumped out; at the same time fresh water in sufficient quantities to preserve its equilibrium, is pumped into the bottom of the leaf. Then, at a little before high tide, when the resultant weight of the leaf is only a little greater than its buoyancy, the outer edge of the leaf is raised by the boat's tackle until there is no weight on the hinges. The hinge-pins are then removed, the leaf is lifted and towed out of the way of traffic, and the air chamber is unwatered by compressed air. The leaf, now floating on its straight side, which is the heavier, has heretofore been beached for cleaning and painting, but a small dry dock is now being constructed for this purpose. It is then towed back to the lock, and the air chamber is partly filled with water until the leaf stands almost upright. The boat then lifts it back on to the pintle and the anchors are attached. The operation of removing and replacing the hinge-pins must be done very quickly at the lower gate, as a slight fall in the tide will often change the buoyancy of the leaf so that the lift will be too great for the boat to handle safely. To provide for this emergency a pump is kept at hand for removing the water from the leaf as the tide recedes. The removal of the upper gate in water of constant elevation is a much simpler undertaking.

The guard-gates of the small lock are hoisted by an **A**-frame and tackle on to the lock-wall, where they are cleaned and painted and then lowered back into place.

A pumping plant consisting of two 30-in. pumps with a capacity of about 50 000 gal. per min., one 10-in pump, with a capacity of about 2 000 gal. per min., and a 3-in. sump pump, all electrically operated, has been provided for unwatering the locks. The pumps are located in a well under the admin-

istration building, the intake being through the lock-wall from a sump just above the intermediate gate. The discharge is carried in a concrete pipe supported on the back of the lock-wall and emptying just below the guard-gate. The pumps are installed near the bottom of the well at about the elevation of the lock floor and have vertical shafts. The large pumps are driven by 300-h.p. motors and the 10-in. pump by a 75-h.p. motor, all located over the well on the basement floor. The pump-well is never allowed to fill with water, the seepage being pumped out by the sump pump which operates automatically. The drainage from all the machinery recesses in the lock-walls is carried through tunnels to the pump-well whence it is removed by the sump pump. It usually requires about eight hours to unwater the large lock, and the painting and repairs are usually accomplished in a week.

The lowering of Lake Washington made it necessary to carry all the water from the Lake Washington and Cedar River water-sheds (580 sq. miles) through the canal to the sea. The maximum known run-off from the water-shed is 12 000 sec.-ft. for a period of 24 hours; 6 000 sec.-ft. for a period of 10 days; and 3 600 sec.-ft. for a period of 90 days. In the low-water season the run-off from the land area is at times so small that it does not equal the evaporation from the lake surface. These periods, however, are of brief duration and occur only when extreme drought and high temperature reduce the run-off and correspondingly increase evaporation from the water areas. Owing to the large capacity of the reservoir formed by the lakes and Salmon Bay the control of this variable run-off can be easily taken care of with an allowable fluctuation of 2 ft. by raising the lake levels to about the high-water elevation before the commencement of the dry season and by lowering the level during the winter months to take care of sudden floods. The discharge over the spillway dam is regulated so as not to cause excessive currents below the locks, and the current through the canal above the locks is never troublesome to vessels.

Electric power for the operation of the locks is delivered at the transformer station at the locks at 13 200 volts and is transformed at the lock site to 2 300 volts, 220 volts, and 110 volts, for operating motors and for lighting the grounds and buildings. The lock-gates and valves are controlled from operating stations on the lock-walls. Three stations are provided for the large lock and one for the small lock. All operating machinery is placed in covered recesses in the top of the lock-walls. For replacing broken cables and doing necessary under-water work, a complete diving equipment is provided, and a diver who is ordinarily employed as an operator on the lock force is sent down to make the necessary repairs.

The delay due to broken cables, which almost invariably affects one or more vessels, has been considerable, probably 2 to 4 hours for each break. Although the gate-operating method is otherwise satisfactory the difficulties arising from this feature suggest that if another lock were to be constructed in the future it might be advisable to try the spar and bull-wheel method similar to that used at Panama, which has all the operating mechanism above water.

As noted previously a small dry dock for docking one leaf of the guard-gates at a time for painting and repairs is being constructed back of the north wall of the large lock, a little below the upper service gate. The lock-wall forms one side of the dock and the quay wall one end. The other side is constructed of two rows of round piles and sheet-piles tied together with iron rods, and filled and banked with clay. The other end will be closed by a timber gate-sill and the gate, which consists of a horizontal girder and needles.

The time required for filling the large lock is about 8 min. at medium low tide. If the culverts were somewhat larger, this time would be decreased. It may be questioned, however, whether this decrease in time would be commensurate with the necessary increase in cost of the original construction of larger valves and operating mechanism.

As constructed, the number of laterals in the lower half of the lock is greater than in the upper half. The idea was that this would result in a more nearly uniform filling of the lock. In practice, it appears that the filling takes place faster in the lower end, resulting in a slight up-stream current in the lock. This has not been particularly troublesome, but in a new design the number of laterals in the lower half should probably be made the same as, or even less than, in the upper half.

Also in a new design a larger salt-water sump and outlet would undoubtedly be provided as suggested under the discussion on "Salinity", and if part of the water now passing over the spillway were carried through the dam at as low an elevation as practicable some additional salt would be carried off.

The concrete work of the locks and dam has now been subjected to the action of both fresh and sea water for more than ten years and does not show any appreciable deterioration, except that the face of the lock-walls has been worn off to a limited extent by the rubbing of vessels and other craft. It is true that it is not subjected to extreme changes of temperature, nor to the action of ice, but its general excellence is such as to make it interesting to record in more detail its composition and method of placing.

The mixture used was practically 1:3:6. The sand and gravel were obtained by washing the material from a bank with sea water, using screens of the desired mesh. The sand was well graded, but the gravel had an excess of fine material and generally in mixing 2½ parts of sand to 6½ parts of gravel were used. The proportions, 1:2:6, would give nearly the maximum density for the materials used. The cement was manufactured by the Superior Portland Cement Company and tested in accordance with specifications of the U. S. Bureau of Standards, except that 85% instead of 75% by weight was required to pass the 200-mesh sieve. The concrete was mixed in 1½-cu. yd., Smith, tiling dump mixers, 2 min. being required for each batch. Each batch was dumped into bottom dump buckets which were carried on flat cars about 1 000 ft. to the site. The buckets were picked up by crane and lowered so as to leave just enough clearance for the doors to open before the batch was emptied. All concrete was spread out in layers and carefully spaded, especially next to the form. No tamping was required. The con-

sistency of the mixture was such that a man would sink about half way to his knees. No parts of the work were subjected to the action of sea water until the concrete was thoroughly cured.

Salinity

The up-stream under current of salt water and the down-stream surface current of fresh water during lockages due to the difference of density have been referred to previously. The flow of salt water into the upper pool would eventually turn the lakes into a salt-water reservoir if some means were not provided to return the salt to the lower pool. To meet this difficulty a salt-water basin about 2 000 ft. long and 200 ft. wide, with its bottom 4 to 8 ft. below the elevation of the upper miter-sill was dredged immediately above the lock. From the lower end of the basin a pipe 30 sq. ft. in cross-section extends to the spillway dam. Its outlet is 4 ft. below high tide, which gives a maximum head of 11 ft. and thus forces the salt water from the bottom of the basin through the drain into the lower pool. The flow is continuous and at the rate of 100 to 200 cu. ft. per sec. This discharge of salt water from the upper pool is increased by opening the valves in the culverts at times when the lock is not busy and when water can be spared for that purpose. These measures are ordinarily effective, but when there are a number of consecutive lockages during the dry season the salt-water basin becomes filled with sea water faster than the drain carries it off and it then overflows and some salt water flows up the canal into the bottom of Lake Union. The lake acts as another settling basin, preventing the flow of salt water into Lake Washington. It has been found that the sea water that enters Lake Union tends to distribute itself throughout the waters of the lake, owing to natural diffusion, disturbances caused by currents, changes in temperature, and the passage of boats. However, it is believed that the greater quantity of the sea water will remain below the 40-ft. level.

To determine accurately the extent of this infiltration into the lakes, the services of Professor E. Victor Smith, of the Department of Chemistry, of the University of Washington, were engaged to conduct a continuous set of observations at different points in the canal and lakes and these have been carried on since the canal was opened in 1917. Professor Smith and his colleague, Professor Thomas G. Thompson, have been much interested in this work and have recently presented a paper* before the American Chemical Society giving the results of their studies. What follows is extracted from that paper.

The device generally used for sampling the waters for analysis is a reversing water bottle of the Eckman type, with attached reversing thermometers. For determining the quantity of chlorine present in the water, various methods were studied. Attention was given to conductivity and specific gravity methods, but results of considerable variation were obtained, owing to difference of temperature and contamination from industrial plants along the waterway. The Mohr method gave the most concordant results. Parts per

* "The Control of Sea Water Flowing into the Lake Washington Ship Canal," *Industrial and Engineering Chemistry*, Vol. 17, No. 10, October, 1925, p. 1084.

million of chlorine were determined, which was considered to give a relative measure of the concentration of sea water. These results were checked by running, on several occasions, complete analyses of the water from Lake Union and comparing the data with those secured from sea water, calculating for the dilution of the water from the lake. Samples were also occasionally checked in the laboratory by the Volhard method.

As from one hundred to two hundred analyses were necessary for each study it was almost impossible to collect samples and transport them to the laboratory. Accordingly, an improvised laboratory was set up on the U. S. Motor Boat *Orcas*, and as fast as the samples were brought aboard they were analyzed by pipetting off 100 milli-litres and titrating. Two standard solutions of silver nitrate of different concentrations were used, one for water of low salinity and the other for water of high salinity. Possible discrepancies due to temperature changes were within the ordinary experimental error, there seldom being a variation greater than 7° cent. in the temperature of the water throughout the entire year.

The chlorine content of the waters at various depths in and near the locks is shown in Table 2. All the samples from which the data were secured were collected the same forenoon. Series 1 shows, in parts per million, the chlorine obtained from the water taken at various depths in Puget Sound about 1 mile from the entrance to the locks. The effect of drainage water from the land into the sea is distinctly indicated in this series of analyses, as the concentration of chlorine on the surface is decidedly lower than that secured at greater depths.

TABLE 2.—Parts per Million of Chlorine at Various Depths at Stations 1 and 2.

(Puget Sound and in the locks.)

Depth, in feet.	Series 1.		Series 2.		Series 3.		Series 4.	
	Temperature, in degrees centigrade.	Parts per million, *Cl.*	Temperature, in degrees centigrade.	Parts per million, *Cl.*	Temperature, in degrees centigrade.	Parts per million, *Cl.*	Temperature, in degrees centigrade.	Parts per million, *Cl.*
Surface...	13.3	14 450	14.2	144	....	68	14.5	66
10	....	.....	12.2	8 681	13.9	64	14.2	117
20	....	.....	11.0	13 760	13.6	106	14.0	244
30	....	.....	10.8	14 340	13.3	663	13.0	573
40	....	.....	10.7	14 520	12.4	5 355	12 4	7 180
50	10.4	15 900	....	.....	....	.....	....	.....
100	9.5	16 590	....	.....	....	.....	....	.....
200	8.9	16 600	....	.....	....	.....	....	.....
300	8.9	16 640	....	.....	....	.....	....	.....
400	8.9	16 640	....	.....	....	.....	....	.....
500	8,8	16 730	....	.....	....	.....	....	.....

The collection of samples under Series 2 was started 10 min. after the lock was opened to the sea and during an ebbing tide. The samples were taken at a point midway between the two lock-gates. The data indicate how

completely the sea water replaces the fresh water in the locks when they are opened to the sea.

Immediately after securing the samples of Series 2, those of Series 3 were obtained on the fresh-water side of the upper lock-gate. On the completion of this operation the lower gate was closed to the sea, the water in the lock raised to the level of the upper pool, and the upper gate opened. As soon as the currents in the lock had ceased, samples were taken from the same position as that from which those of Series 2 had been secured. The results, given under Series 4, demonstrate that the sea water had largely disappeared by flowing into the basin above the lock. The sample taken from the bottom of the lock below the elevation of the upper miter-sill was the only one showing a high degree of salinity.

It is impracticable to show all the data secured during the eight years that this investigation has been under way. From those presented in Table 2 it is obvious that the waters of the canal near the locks undergo constant change. However, as the distance from the lock increases, the analyses of the water indicate fairly constant conditions. For example, in Lake Union the change is relatively slow, so that remarkably constant results may be obtained from month to month, especially at the greater depths. Samples were taken at the different stations, indicated in Fig. 1, at every 10 ft. Table 3 gives the relative distances of the respective stations from the locks.

TABLE 3.

Between stations.	Distances between the various stations, in feet.	Distance from locks (Station 2), in feet.
1 and 2	.....	.....
2 and 3	1 500	1 500
3 and 4	3 650	5 150
4 and 5	2 500	7 650
5 and 6	5 400	13 050
6 and 7	1 650	14 700
7 and 9	3 000	17 700
9 and 10	5 000	22 700
10 and 11	4 500	27 200
11 and 12	10 500	37 000

In Table 4 are given analyses obtained on February 21, 1920, October 8, 1921, and June 10, 1922, at various depths and at the different stations. The results of February, 1920, are illustrative of minimum conditions of salinity, although, in March, 1921, the entire system was cleared of sea-water contaminations. The worst conditions of the eight years were encountered in October, 1921, after a very dry summer, and were due in part also to the clogging of the salt-water drain for a time by a sunken log. This shows very well the ease with which the waters of the canal and lakes might have become contaminated if proper control had not been provided. The data for June, 1922, indicate the effect of drainage resulting from winter rains and spring freshets. In Fig. 10 the logarithms of the parts per million of chlorine for the maximum and average conditions are plotted for the various depths at the different stations along the waterway.

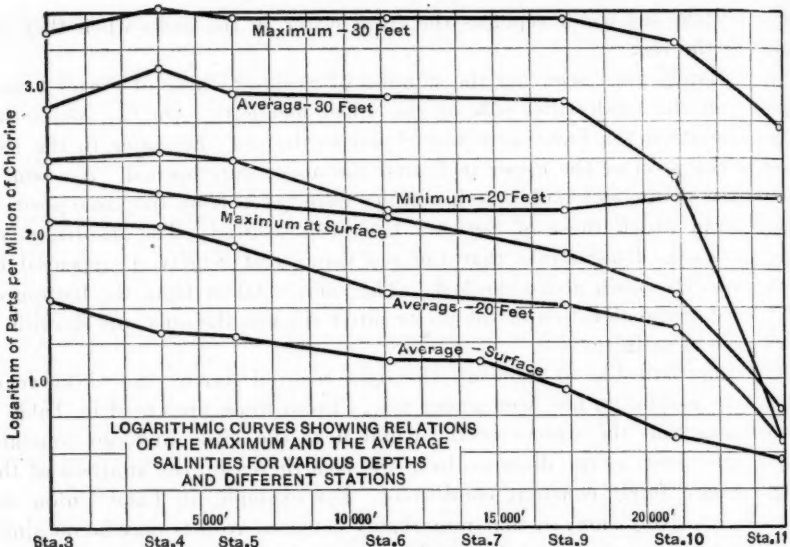


FIG. 10.

Rather peculiar results for maximum conditions at depths of 20 ft. and 30 ft. and for average conditions at 30 ft. are indicated by the curves in Fig. 10 at Stations 3 and 4. The former, being nearer the locks, should show a

TABLE 4.—PARTS PER MILLION OF CHLORINE, SHOWING MAXIMUM AND MINIMUM CONDITIONS OBTAINED AT VARIOUS DEPTHS AND DIFFERENT STATIONS.

Station.	Date tested.	Depth, in Feet.							
		0.	10.	20.	30.	32.	40.	45.	50.
3	Feb. 21, 1920	.....	.....	.....	.....	.....	.....	.....	.....
	Oct. 8, 1921	244	237	310	2 315	.....	7 800	9 075	.....
	June 10, 1922	10	9	33	325	.....	.....	.....	.....
4	Feb. 21, 1920	16	16	26	55	130	.....	.....	.....
	Oct. 8, 1921	193	198	360	3 555	.....	.....	.....	.....
	June 10, 1922	6	8	13	600	.....	.....	.....	.....
5	Feb. 21, 1920	16	15	16	14	.....	.....	.....	.....
	Oct. 8, 1921	169	171	315	2 920	.....	.....	.....	.....
	June 10, 1922	3	4	7	75	.....	.....	.....	.....
6	Feb. 21, 1920	9	12	15	14	.....	.....	.....	.....
	Oct. 8, 1921	128	137	135	2 940	3 125	.....	.....	.....
	June 10, 1922	3	3	4	20	28	.....	.....	.....
9	Feb. 21, 1920	7	9	9	9	.....	4 149	5 040	.....
	Oct. 8, 1921	70	111	145	2 875	.....	4 710	5 830	.....
	June 10, 1922	4	3	5	10	.....	420	3 995	.....
10	Feb. 21, 1920	3	3	3	3	.....	3	.....	.....
	Oct. 8, 1921	36	37	178	1 985	.....	2 507	.....	.....
	June 10, 1922	3	3	3	4	.....	4	.....	.....
11	Feb. 21, 1920	3	3	3	3	.....	.....	.....	.....
	Oct. 8, 1921	6	14	173	505	.....	.....	.....	.....
	June 10, 1922	3	3	3	3	.....	.....	.....	3
12	Feb. 21, 1920	3	.....	.....	.....	.....	.....	.....	3
	Oct. 8, 1921	3	.....	.....	.....	.....	.....	.....	3
	June 10, 1922	3	.....	.....	.....	.....	.....	.....	.....

greater degree of salinity, but much of the data collected show that Station 4 generally gave higher concentrations. This is probably the result of counter-currents.

The following conclusions were reached:

(1) That the concentration of sea water in the Lake Washington Ship Canal is dependent on the amount of rainfall, the number of lockages, the proper functioning of the salt-water drain, and the methods of disposal of surplus water.

(2) That if the surplus waters are conducted as much as possible through the lock culverts, the degree of salinity can be maintained at a minimum far lower than if the surplus is permitted to run wholly over the spillways.

(3) That greater efficiency could undoubtedly be obtained if the salt-water basin were enlarged in length and width, or depth.

(4) That Lake Union serves as a secondary salt-water basin and prevents the contamination of the waters of Lake Washington.

(5) That storage of surplus water of the winter and spring months is used as far as practicable for drainage through the lock-culvert valves through the dry season which aids materially in decreasing the salinity of the canal system.

The discharge through the spillway gates of the dam is very small between February and October, and from July to October there is usually no discharge, except that used for lockages, passing through the salt-water pipe and over the fishway. The large lock is usually filled from 200 to 400 times per month and the small lock from 500 to 1 000 times.

Traffic

The canal has been in operation since August, 1916, and there has been a substantial increase in traffic and in the value of commodities from year to year. The commerce for 1917 was 872 359 tons of freight, valued at $4 254 202 and, in 1920, 1 368 898 tons, valued at $15 567 829. In 1925, it amounted to 2 418 613 tons valued at $14 712 520, and consisted of 1 192 540 tons of floated and barged timbers, valued at $6 372 380; 898 892 tons of sand and gravel, valued at $898 892; 174 677 tons of lumber products valued at $2 649 020; 54 634 tons of fuel oil, with a value of $524 486; 23 350 tons of refined oil, valued at $1 097 800; 23 433 tons of coal and coke, valued at $187 764; 19 950 tons of hogged fuel, with a value of $11 970; and 31 137 tons of miscellaneous, valued at $2 970 208. The number of passengers carried was 49 792. In addition, there were carried by general ferries 269 031 tons and 1 401 037 passengers in 1925.

The bulk of the traffic passing through the locks is between points on Puget Sound, Salmon Bay, Lake Union, and Lake Washington, but lumber is shipped through the canal to points on the Pacific and Atlantic Coasts, and to foreign countries, and there are considerable imports of logs, coal, etc., from Canada.

Vessels using the canal are increasing in size as well as in numbers. During 1917, 22 392 vessels with a net registered tonnage of 591 310 passed through the locks; in 1920, there were 27 377 vessels, with a net registered

tonnage of 1 120 591; and, in 1925, this had increased to 35 433 vessels, with a net registered tonnage of 2 279 131. The largest number of vessels passing through the large lock in a single day has been 104, which included small boats, as the small lock was closed. The greatest number of vessels passing through the small lock in a single day has been 157.

Practically all freight passes through the large lock and about one-half of the total consists of logs, floated loosely between boom sticks in rafts. At times, there has been some congestion of traffic, and an examination has been authorized by Congress with a view to determining the necessity for an additional lock. It does not at present appear, however, that the capacity of the lock has been fully reached. Delays to boat traffic could be largely reduced by restricting the passage of log rafts to the night time when other traffic is normally at a minimum. If this were done a mooring ground, safe from storms, should be provided in Shilshole Bay near the entrance from Puget Sound.

Greater efficiency of operation could be obtained if the logs were made up in sections of deeper draft or transported in barges. During 1924 there was some traffic in logs transported in barges from British Columbia, 400 000 to 800 000 ft. B. M. being carried in each barge. Logs transported in this way are not subject to delay on account of stormy weather and for long hauls transportation by barge is found quite economical. From Puget Sound ports, however, where the haul is relatively short, the additional expense for loading, upkeep of barge, etc., would more than offset the expense saved in towing and loss by storms, and if a restrictive regulation of this kind were put into effect it would tend to increase the price of logs entering the canal from all Sound ports.

On account of the current previously referred to caused by the meeting of fresh and salt water in and near the locks, and the current from the spillway dam when the gates are open, a material saving in time would be accomplished if the use of only powerful tugs were permitted for bringing heavy tows into the locks.

The small lock, which is 30 by 150 ft., was designed primarily to accommodate small craft, such as tugs without tows, pleasure boats, etc., and the dimensions for this class of traffic are sufficient. The present increase in traffic, however, shows the need of a lock of greater width, capable of handling barge traffic when the large lock is unwatered for annual repairs, as well as relieving the latter from some of this traffic at other times. A lock, 34 to 38 ft. wide and 160 to 170 ft. long, would handle the present traffic more efficiently than the lock as built.

Results

The beneficial results of the construction of the canal may be generally summarized, as follows:

(*a*) It has opened to commercial and industrial use a fresh-water harbor, lying in and adjacent to the City of Seattle, with an area of 25 000 acres and a shore line of 100 miles. This harbor consists of three distinct bodies of

water, Salmon Bay, Lake Union, and Lake Washington, all standing at the same level and connected by free and unobstructed channels. This harbor is perfectly protected from storms and free from tidal fluctuations and currents, and, with the salt-water harbor on Elliott Bay, affords harbor facilities which are possibly unsurpassed in the United States.

(*b*) A large and important commerce has already been developed, and Lake Union, located less than a mile from the business center of Seattle and accessible to both rail and deep-water shipping, with depths ranging from 40 to 50 ft., and an area of 905 acres, seems logically destined to become a great distributing center and warehouse district.

(*c*) In the fresh water of these lakes untreated pile and timber structures may be maintained free from destruction by marine wood borers, the marine growth on ships' bottoms is killed by contact with fresh water, and ships may be more economically loaded and unloaded on account of having a fixed water level.

(*d*) The lowering of Lake Washington and the diversion of Cedar River from the Black and Duwamish Rivers into the lake has practically ended the annual floods which formerly visited the Renton, Duwamish, and Sammanish River Districts, and former swamps have been converted into favorable sites for industries, the principal of these tracts being in the Renton District at the south end of the lake, easily accessible to all the railroads entering Seattle, and in the Bothell District at the north end of the lake.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE WORK OF THE PORT OF SEATTLE, WASHINGTON*

By George F. Cotterill,† Esq.

The "Port of Seattle" must be considered in a dual sense: First, in its broadest meaning as the great harbor of the North Pacific, with a continent for a hinterland and an outlook on the ocean highways, strategically located by Nature and equipped by development to serve the needs of world commerce; and, second, in its lesser scope as the official designation of a municipal corporation charged with certain powers and duties in the development and service of commerce in the Harbor of Seattle.

Strategic Location of Seattle

In geographical terms, the Port of Seattle may be described as situated on Elliott Bay, on the eastern and continental side of Puget Sound approximately 125 miles from the open Pacific Ocean, passage being made through the Strait of Juan de Fuca, Admiralty Inlet, and Puget Sound. The Strait is about 15 miles wide at the Pacific entrance between Cape Flattery and Carmanah Lights, and extends eastward, with an average width of more than 10 miles, between the State of Washington on the south and Vancouver Island on the north, for about 50 miles to Race Rocks, on the Canadian side, there expanding to the northeast and continuing eastward, with an average width of 18 miles, for 30 miles, connecting northward through Georgia Strait into the Canadian waters east of Vancouver Island, thence back to the North Pacific and inside passages of Southeastern Alaska, also connecting southerly through Admiralty Inlet into Puget Sound extending for 75 miles into the heart of Western Washington. The Strait of Juan de Fuca extends eastward from Cape Flattery to Point Wilson, about 85 miles, there connecting southward into Admiralty Inlet and Puget Sound, which have a width from 3 to 5 miles, continuing for 40 miles to the Port of Seattle and 30 miles farther

Note.—Written discussion on this paper will be closed in December, 1927.
* Presented at the meeting of the Waterways Division, Seattle, Wash., July 15, 1926.
† Civ. Engr. and Port Commr., Seattle, Wash.

south to the Port of Tacoma. Both Strait and Sound are of ample depth, varying from 150 fathoms at Cape Flattery to 25 fathoms minimum at the Admiralty Inlet entrance to Puget Sound, which has similar variations of depth from 150 to 25 fathoms minimum at the opening into Elliott Bay—the Port of Seattle. Here is a natural harbor, with 9.7 miles of shore line, including 3 800 acres of surface area, very deep and free from natural obstructions or dangers throughout its entire extent—a land-locked bay indenting the continent from an inland sea, the open hand of a mighty arm of the ocean extended 125 miles inward for the largest service of commerce with utmost safety and convenience.

Viewed from the continental hinterland the Port of Seattle demonstrates the most strategic location, nearest to central for seven, and the focal Pacific port of concentration for three, transcontinental railway systems. Two northern routes traverse Canada from the Atlantic to the Pacific, finding a Pacific outlet at the Port of Vancouver, but with rail and water connections to the Port of Seattle, 150 miles to southward. Likewise, two routes from the Middle West and the Pacific Northwest find their way down the Columbia River to the Ports of Portland and Astoria in Oregon and to the new Ports of Vancouver and Longview in Washington, but these railway systems have also joint ownership and common-user operating rights over the 150 miles of almost water-grade railway northward from the Columbia River to Puget Sound and the Port of Seattle. Midway between these four connecting routes of Canada and the Columbia River are the three great American northern transcontinental systems, covering the great Northwestern Empire from the Great Lakes and the Upper Mississippi Valley to the Pacific slope—these converge at the Port of Seattle.

The Climax of a World Quest

There is a historical background worthy of note before turning from the geographic picture of the Port of Seattle and passing to detail consideration of its development and progress. Columbus, seeking the westward route "to India and Cathay", found it blocked by two continents linked by an isthmus barrier. Diaz and De Gama demonstrated the southeast passage *via* the Cape of Good Hope to India and the East Indies; to which Magellan soon added his momentous contribution of the southwest route by the Straits which bear his name, to the Indies and the first circumnavigation of the globe. Balboa meanwhile crossed the Isthmus of Panama and discovered the "South Sea", along which Spain soon extended her conquests southward to Peru and northward to the Gulf of California. Then began the search for a "Northwest Passage" to China around the supposed north coast of, or through, North America. It was first pressed by England, then by France in the North Atlantic. Meanwhile, Spain was extending her knowledge of the North Pacific. Two Philippine ships in 1584 and 1595 made a dim record; in 1602, Viscanio from Spanish Mexico went north to the Bays of San Diego and Monterey, and on to 42° Latitude—the northern boundary of California. In 1603, Martin Aguilar reached 43°, named Cape Blanco and claimed to

have discovered the mouth of a large river. Globes and maps of this period began to indicate an indefinite "Strait of Anian", probably the expression of a hope of an opening eastward, somewhere north of known discoveries and fading away into the unknown.

Discovery lagged after the first third of the Sixteenth Century, while the nations of Europe turned to exploration, development, and extension of their domains and claims on the continent that barred the way to the Orient. On the Pacific side, Spain coasted north from Mexico, Cabrillo in 1542-43 exploring the southern coast of California and Ferrelo in 1543 as far north as the Oregon coast. The name, California, is first found in a Spanish romance of 1510, described as being "on the right hand of the Indies, very near the terrestrial paradise", hence it was applied by Cabrillo to the coast he explored in 1542. Lower California was considered an island and so mapped up to 1721.

The war between Spain and England in the Elizabethan era was carried to all the seven seas, over which Spain held sway as the mistress of world commerce. Bent on a mission of war destruction, Sir Francis Drake left England in December, 1577, went down the east coast of South America, through the straits of Magellan (September, 1578), plundered Spanish possessions in Chili and Peru, seized ships, and demoralized Spanish Pacific commerce all the way up the coast to Mexico and California, particularly intercepting their treasure ships from the Philippines and East Indies en route to Panama and the Atlantic. Laden with rich prizes of war—after landing and refitting at Drake Bay, near San Francisco—Drake sought a shorter route to England by way of the supposed "northwest passage". He proceeded north, up the California and Oregon and perhaps the Washington coast, seeking an opening or trend of the coast to the eastward. Whether he stopped at 43° North Latitude or continued to 48°, just south of the great strait opening into Puget Sound, accounts differ. It was late summer of 1579, and Drake feared that even if he continued north and found an eastward passage it would only lure him into the frozen regions of the North Atlantic. Hence, he turned westward across the Pacific, doubtless making the first trans-Pacific voyage by the northern route, and continued by way of the Indian Ocean and the Cape of Good Hope to England in 1580.

At this stage there was published by an English author the purported memorandum of Michael Lok of a conversation at Venice in 1596 with an old Greek pilot, Juan de Fuca. By this third-hand authority there came the story which pointed the finger to an unknown Puget Sound as the Pacific portal of the Northwest Passage—the American exit of the trade route to the Orient. Juan de Fuca was reported as giving the story of a voyage in 1592, sent out by the Viceroy of Nova Spania (that is, Western Mexico).

For a century and a half the search for the Northwest Passage and the trade route to the Orient languished. Before the middle of the Eighteenth Century came the awakening. In 1728, the Danish explorer, Bering, in the service of Russia, demonstrated that Asia and North America were two continents, with a sea and strait between, connecting from the Pacific to the

ice-bound Arctic. Also, in 1741, Bering coasted North America down to 54° 40′. The fur trade of the North Pacific developed and the merchants of all nations sought to enter it. England, meanwhile, had taken over the Portuguese possessions in India and Southeast Asia and was securing trade footholds in South China. In the American Continent her Hudson's Bay Company was stretching westward up great rivers toward the mountains and the Pacific that lay beyond. The only available market for the North Pacific furs were the Ports of South China and India. England reopened the book of discovery after nearly two centuries, at the page turned down by the Juan de Fuca story of 1592, and offered a $100 000 reward for the demonstration of the "Northwest Passage". With this incentive, Captain James Cook—just 200 years after Drake's vain effort of 1579—sought the prize. It was the third and last voyage of this greatest of British navigators. Sailing on March 7, 1778, from the Sandwich Islands—which he had discovered and named on a previous Pacific voyage—he sighted the coast at the North California or Spanish boundary and followed it northward to Nootka Sound on the west coast of Vancouver Island, observing neither the Columbia River or other opening in the Coast line, having been driven to the open ocean by a gale.

Fourteen years after Cook failed where Drake fell short two centuries before, the lot fell on Captain George Vancouver to solve the problem and dissolve the mystery of three centuries. The British Admiralty having accepted the conclusion recorded by Cook, the fur trade developed from the Nootka base, with the South China and East India market, participated in by British and American merchantment after the treaties of 1783 closing the Revolutionary War. Suddenly, in 1790, the news was brought to London by Captain John Meares, of the Spanish seizure of his two British merchantmen at Nootka, with their cargoes of furs. The menace of war was aroused, but soon there was an Anglo-Spanish treaty, including apologies and reparation. More important than his news of seizures and demand for reparation, however, Meares had brought to London the news of new discoveries in the North Pacific, and particularly that "John de Fuca's Strait" really existed, despite Cook's 1778 record of non-existence. Meares confronted the 1790 London world of geography with a map entitled "A Chart of the Interior Part of North America; Demonstrating the Very Great Probability of an Inland Navigation from Hudson's Bay to the West Coast". The map showed "John de Fuca's Straits" entering from the ocean eastward to an undefined open end; a dotted line around to the northward and back into the Pacific Ocean north of Nootka, marked "Sketch of the Track of the American Sloop *Washington* in 1789"; an indication of "The Sea" to the eastward; a short section of the "Oregon River" in the interior, with dotted lines indicating probable connection to the inland sea and straits; also an intricate system of lakes and rivers covering the Continent to the north and eastward. By this Meares map and information, all the doubts and hopes of a Northwest Passage as the great trade route from Occident to Orient were renewed. The British Admiralty acted promptly, deciding to combine with the Naval Expedition to adjust

the Spanish seizures at Nootka, a final and thoroughly equipped scientific solution or dissolution of the North Pacific mystery. Hence, the Vancouver Expedition of 1791-93.

Captain Vancouver faithfully discharged his mission, entered the Straits of Fuca, spent the season of 1792 in a survey and charting of the right-hand coast line of the Continent, testing out every inlet of the intricate ramifications of the inland sea, east and south and north and northwest, until at last he came out into the Pacific north of Vancouver's Island. The high point of this great voyage of discovery was reached May 26, 1792, when Vancouver rounded the point into Commencement Bay, the Harbor of Tacoma. The record reads:

"Towards noon we landed on a point on the eastern shore * * * round which we flattered ourselves we should find the inlet take an extensive easterly course. This conjecture was supported by the appearance of a very abrupt division in the snowy range of mountains immediately south of Mt. Rainier, which was very conspicuous from the ship, and the main arm of the inlet appearing to stretch in that direction from the point we were then upon.

"We here dined, and although our repast was soon concluded, the delay was irksome, as we were excessively anxious to ascertain the truth, of which we were not long held in suspense. For having passed round the point, we found the inlet to terminate here in an extensive, circular, compact bay, whose waters washed the base of Mount Rainier".

Thus ended the search for the "Northwest Passage". The dream was dissolved into the reality of a North Pacific "Mediterranean", with sheltered harbors destined for great ports, ready to serve the continents through the giant gateway of the spacious straits to the world highway of the greatest ocean.

The age of steam and steel, dawning a half-century after Vancouver, made possible the realization of the Northwest Passage and the world's greatest trade route by the steamship and the railway. On Vancouver's Puget Sound of 1792, the story of the Port of Seattle was started sixty years later. It is the realization of a vision, the fulfillment of destiny.

Development of the Port of Seattle

The Port of Seattle, in its limited official scope, is a municipal corporation, covering the territory of Seattle and its surrounding County of King, organized and operated under authority of Washington State Law in 1911.

Prior to this public port organization, Seattle had developed, during the preceding forty years:

(1) A large "mosquito fleet" commerce covering the intricate ramifications of Puget Sound and tributary inland waters, serving the suburban vicinity and the industrial and agricultural communities of the Puget Sound region, which always included more than half the population of Washington;

(2) A coast-wise, inland waters, and deep-sea commerce with British Columbia and Alaska, and with the California port; and,

(3) A trans-Pacific commerce initiated by the Great Northern Railway in connection with a Japanese steamship company in 1896, and, later, with its own great American cargo steamer, the *Minnesota*. There was occasional service by British companies in the Oriental trade; also,

(4) An industrial commerce in lumber to South American, Australian, and other Pacific markets, and some small share of the grain shipments from the Inland Empire to the United Kingdom and Europe.

The port facilities for the handling of this growing volume of commerce prior to 1911, had developed from the original Yesler Mill Wharf of 1853, around which the city was founded, to a considerable magnitude by 1889, when an $8 000 000 conflagration destroyed the central business section, including the entire water-front, with perhaps $1 000 000 of wharves and warehouses. All were rebuilt on larger scale in 1889-90 and expanded to meet the growing needs of commerce. Before 1911 and public port organization, there had been developed about thirty wharves and piers, owned by railroad and steamship companies, industrial concerns, and a few commercial dock companies, all privately owned and operated, under slight degree of regulative State authority in some cases.

The extent and growth of these port facilities prior to 1911, may be gathered from the statistics of 1910 as compared with 1890 as given in Table 1.

TABLE 1.

	1890.	1910.
Seattle population	42 837	233 194
Water-borne tonnage	430 260	860 000
Value of shipments	$48 000 000	$96 091 040

During this 20-year period, while the population increased fivefold, the volume of port commerce only doubled. It was evident to all studious observers for several years prior to 1910 that commercial development was not keeping pace with the rapid growth of the city. It became apparent that the port facilities and equipment were inadequate to attract and hold increasing commerce. The Panama Canal was approaching completion, and it was plain that it would mean a re-adjustment of routes of commerce. Looking north and south, Seattle saw every Pacific port from Vancouver to San Diego, planning great port improvements, backed by the public credit of their communities. With freight rates from the interior on a practical parity to all Pacific ports, and trans-Pacific ocean rates identical from all to Oriental ports, with a likelihood of similar parity in intercoastal traffic with the oncoming Panama régime—it was clear that those ports which offered commerce the most adequate facilities and efficient service at the lowest cost would hold their own and go forward in the new era of world commerce which impended with the opening of the Panama Canal; and, *contra*, that any port which failed to prepare for the new era would find itself left in the race.

Seattle girded herself to prepare for this new era. It was obvious that private investment could not be induced or attracted to meet the situation. With tariff rates, necessarily fixed by competition with other Pacific ports, all backed and supported by public credit and taxations, as required, there could

be no profit in constructing port facilities in advance of their need, sufficient to invite investment of private capital in common-user water-front terminals. It was absolutely necessary that there be authority for public port organizations in the State of Washington, if it was to hold its own with the public ports of British Columbia, Oregon, and California.

Civic committees besieged three State Legislatures, in 1907, 1909, and 1911, before gaining the required general law of port authorization. Briefly, the law enables any harbor district, including part or all of a county, to organize by vote of the people, as a port municipal corporation, endowed with the public credit up to 3% of the assessed total of taxable resources of the district, with taxing power for debt charges and up to 2 mills per year for any port purpose. The port is managed by a commission of three, one elected annually for a 3-year term. Port development must proceed on a comprehensive plan, with specific units of improvement, adopted by majority vote of the electors of the district, and bonds for indebtedness by a 60% vote.

The Port of Seattle was the first to avail itself of this authority and was organized in September, 1911. Tacoma, Bellingham, Everett, Gray's Harbor, Olympia, Port Angeles, Port Townsend, Longview, Vancouver, and several smaller harbor cities of Washington have applied the same general law.

Progress of Port of Seattle

Space forbids the detail description of the accomplishments of the Port of Seattle in its fifteen years of history. Seven port terminals, and equipment affording service for the varied needs of commerce, have been provided at cost of about $9 000 000, with an eighth terminal site purchased for $600 000, and an area of State land valued at $600 000, dedicated free for port industrial sites. Most of the construction was done before World War prices prevailed, and even with full allowance for depreciation, the present (1926) value of port properties and improvements is at least $12 000 000. These terminals include:

(1) Salmon Bay Terminal; headquarters for fishing fleet; industrial sites, etc.; present value	$600 000
(2) Smith's Cove Terminal; two great ocean piers, 40 and 41; largest in world; special equipment for heavy lifts, facilities for oils, molasses, etc.; industrial area adjoining; present value	3 400 000
(3) Bell Street Terminal; central water-front wharf; four-story concrete warehouse and office buildings; six-story, 80 by 104-ft. cold storage annex; present value	960 000
(4) Marion Street Ferry; single slip landing; two-story business and office frontage; present value	110 000
(5) Stacy-Lander Terminal; double wharf, with transit sheds and two-story warehouse frontage; four-story concrete warehouse, 78 by 318 ft.; present value	1 050 000
(6) Hanford Street Terminal; 1 000 000-bushel grain elevator, conveyors, and water-side loading facilities; wharf and two-story transit shed; present value	1 100 000

(7) Spokane Street Terminal; wharf and two-story transit shed, 90 by 882 ft.; two-story salmon warehouse, 147 by 514 ft.; fish freezing and storage facilities, 121 by 172 ft., and 101 by 109 ft.; ice storage, 5 000 tons; cold storage building, seven stories, 147 by 195 ft.; compressor, refrigerating, and ice plant; present value $1 300 000

(8) Unit No. 15; central water-front site, 26 acres; cost, $600 000; present value..... 1 800 000

(9) Canal waterway (vacated); industrial sites..... 600 000

Since these public port facilities have been added to the service of commerce the development during fifteen years may be summarized as in Table 2.

TABLE 2.

	1910.	1925.
Seattle population	237 174	400 000*
Water-borne tonnage	860 000	7 995 083
Value of shipments	$96 091 040	$730 668 972

* Estimated.

Since the organization of the Port of Seattle, the population has almost doubled, while the volume of water-borne commerce has increased eightfold. Manifestly, this remarkable commercial progress did not result entirely through the efforts and operations of the public port corporation, but it is capable of utmost demonstration that the organized Port of Seattle, sustained by the public credit, made possible the provision of modern port facilities, attracting new and varied forms of commerce, presenting a community front to the whole world of trade, and stimulating the older forms of private initiative to expansion and larger activities. It has been the policy of the public port corporation not to supplant but to supplement existing facilities; to plan and build in advance of need and especially to develop new lines of commerce to swell the community total.

All the public port facilities, except the second and larger Pier 41 of the Smith's Cove Terminal, were planned, constructed, and ready for service by 1914, the year of the completion of the Panama Canal. The Port of Seattle was prepared and equipped to meet the new era of re-adjustment of world routes of commerce; but the clouds of war suddenly impended and four years of world conflict absorbed all energies. Normal commerce was destroyed or brushed aside, but an abnormal war commerce, meeting emergency war needs, sprang into existence. Shipping from the Orient, the East Indies, and India, driven from its normal Suez route to Europe by the torpedo menace in the Mediterranean Sea and South Atlantic Ocean, turned eastward to the Pacific ports of America and the protected routes eastward under neutral auspices. When the United States entered the war in 1917-18, the volume of war supply commerce was vastly augmented. The Port of Seattle, with its strategic location, new port facilities, and adequate equipment, seemed as if providen-

tially prepared for the service of this abnormal period. Its commerce suddenly swelled to a volume of foreign trade second only to that of the premier American port, New York. Without suggestion of profiteering, the Port handled this tremendous volume of war-period commerce, with financial return sufficient to carry the entire public investment without need of supporting tax levy, creating also a surplus of almost $1 000 000 which was available and applied to additional port improvements without need of bonds and debt.

Re-Adjustment and Revival

After the war came the slump and re-adjustment period. The war commerce faded away like a dream, and the awakening of America and the world to a commerce rebuilt from the bottom was a slow process. The Port of Seattle was prepared for this new era and progress has been steadily upward. The upward turn came in 1922, since which time the record of water-borne commerce, foreign and domestic, shows as follows:

1922	5 888 044	tons
1923	6 662 889	"
1924	7 886 137	"
1925	7 995 083	"

During this post-war period the public terminals of the Port of Seattle have been maintained and operated with the financial showing noted in Table 3.

TABLE 3.

Port of Seattle.	1921.	1922.	1923.	1924.	1925.
Earnings	$877 521	$920 591	$964 983	$1 059 374	$1 054 201
Expenses	657 493	712 284	737 742	772 298	716 451
Operating gain	$220 058	$208 307	$227 341	$287 076	$337 750
Interest and depreciation	$469 872	$511 864	$467 808	$463 718	$468 055
Net loss	$249 814	$303 557	$240 067	$176 642	$130 315

It may be noted from Table 3 that the sum total of these annually decreasing net investment losses for the five years, 1921-25, approximately equal and offset the net investment surplus accumulated during the preceding five years of abnormal war commerce. Considering, therefore, the entire period from the initiation of operations following the construction period of 1912-13, up to and including 1925, the Port of Seattle has been self-sustaining, carrying the full investment of the public credit, without tax support for any of its operations. The Port is now fully established as a going concern, on a sound financial basis, facing a future of continued expansion to meet the largest demands of commerce with ample resources and credit.

It will be interesting to note the sources and destinations of the 7 995 083 tons of commodities water-borne to and from the Port of Seattle in 1925, as follows:

	Tons.
Domestic (U. S.) Commerce:	
Puget Sound and local	3 408 239
Pacific coastwise	2 223 233
Atlantic ports	594 639
Alaska	395 503
Gulf ports	54 636
Total domestic tonnage	6 676 250
Foreign Commerce (also, Hawaii and The Philippines):	
Orient, Japan, China	404 685
British Columbia	260 327
United Kingdom and Europe	135 663
Hawaii	97 602
Philippines and East Indies	85 262
South and Central America, Mexico, West Indies	42 034
Australia and New Zealand	36 624
Canada, Africa, and all others	264 318
Total foreign tonnage	1 318 833
Port of Seattle, 1925 tonnage	7 995 083

Exports and Imports

Illustrative of the service of the Port of Seattle to the producers and industries of its vicinity and tributary hinterland, the following are some of the commodities exported in foreign trade:

914 683 boxes of apples.
5 622 tons of automobiles.
8 282 tons of canned goods (except fish).
22 692 tons of coal and coke.
13 040 tons of raw cotton.
42 984 tons of fish, canned, pickled, etc.
62 406 tons of flour.
13 361 tons of fodders and feed.
1 339 734 bushels of grain.
12 210 tons of hardware, iron and steel manufactures.
140 325 million ft., B.M., lumber, logs, piling, etc.
4 903 tons of machinery.
9 114 tons of canned milk.
13 932 tons of crude oil and products.
14 746 tons of paper.
3 340 tons of potatoes.
7 767 tons of tobacco.
4 214 tons of wood manufactures (except furniture).
6 795 tons of chemicals.

These are the leading items in a total of 649 581 tons exported (foreign) from Seattle. More than twice this total of these and other commodities are exported (domestic) over Seattle wharves—1 333 082 tons of domestic exports.

Foreign imports through the Port of Seattle in 1925 totaled 669 252 tons, of which some of the leading items were:

11 200 tons of burlap and bagging.
26 954 tons of cement.
91 270 tons of coal and coke.
19 834 tons of coal-tar and products.
2 634 tons of coffee.
3 553 tons of cotton, raw and waste.
4 626 tons of fodders and feed.
1 239 tons of glass.
71 514 bushels of grain (except rice).
5 396 tons of hemp and jute.
12 455 tons of iron and steel.
6 675 tons of lime and plaster.
6 399 tons of minerals.
42 899 tons of oils (bean, cocoanut, peanut, wood, etc.).
30 278 tons of paper.
9 720 tons of peanuts (and other nuts).
9 432 tons of porcelain and pottery.
3 534 tons of seeds and bulbs.
7 736 tons of shingles.
14 597 tons of silk, raw and waste.
2 809 tons of sugar.
13 468 tons of syrup and molasses.
4 613 tons of tea.
4 919 tons of wood, pulp and paper stock.
6 446 tons of carpets and rugs.
1 849 tons of prepared eggs.
$444 089 value of furs and manufactures.

While the domestic imports to and through the Port of Seattle reached a 1925 total of 5 343 167 tons, eight times the foreign volume.

Seattle Realizing Its Destiny

As an evidence that Puget Sound and the Port of Seattle are winning the commerce of the great trade route between the Occident and the Orient, the 1925 record shows that Oriental commerce with American Pacific ports, compared on a tonnage basis, is distributed, as follows: 39.7% through Washington ports; 32.0% through Oregon ports; and 28.3% through California ports.

When the comparison is based on valuation rather than tonnage, the northern route to Puget Sound—shorter by 281 miles than the northern route from San Francisco to Yokohama, and by 1 230 miles as compared to the southern route from San Francisco *via* Honolulu—the strategic location of the Port of Seattle is emphasized by the fact that the silks of the Orient, at the rate of $229 000 000 in 1925, were drawn across the short North Pacific route, whereas less than one-quarter of that volume used the longer route *via* the Golden Gate.

Witness, then, the dream of centuries realized in the actuality of to-day, and the vision of a greater to-morrow. American railway systems converging on Puget Sound and the Port of Seattle serve the old European Occident and the New World of Columbus in trade relations over the shortest Pacific path to the hundreds of millions of the Orient. The fabled "Straits of Anian" of the Sixteenth Century and the demonstration of Puget Sound in the closing Eighteenth Century, have, in the dawning of the Twentieth Century, revealed a "Northwest Passage" beyond the hopes of Drake, the dream of De Fuca, and the achievements of Vancouver and Gray. And the Port of Seattle stands at the gateway to serve world commerce.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

AUTOMOBILE HAZARD IN CITIES AND ITS REDUCTION

Discussion*

By Messrs. William T. Lyle, M. O. Eldridge, C. W. Stark, L. G. Holleran, S. Johannesson, W. Graham Cole, Harold M. Lewis, J. K. Finch, G. G. Kelcey, Raymond A. O'Hara, John A. Miller, Jr., and T. Kennard Thomson.

William T. Lyle,† M. Am. Soc. C. E. (by letter).‡—This paper commands attention not only for its originality, but also for its suggestiveness. It may be classed as a contribution to the literature of city planning. The end sought is the reduction of automobile hazard. The investigations made and conclusions reached are beyond the frontiers of present knowledge.

The analysis is clear and reasonable. The author is careful to define the term, "automobile hazard", at the outset of his paper and this definition must be borne in mind throughout. The hazard resulting from the use of a single automobile varies as $\frac{P}{M}$, the population density; therefore, the variation in the automobile accident rate per unit of population no doubt would be obtained as $\frac{P}{M} \times \frac{R}{P} = \frac{R}{M}$, the automobile density. It is necessary to appreciate these two values in order to grasp the argument for zoning, traffic segregation, street plan, and motor vehicle legislation. Fig. 3§ is not only interesting, but provocative also of many questions. It demonstrates conclusively the value of motor vehicle legislation and of a city plan.

Since the variation in the automobile accident rate per unit of population is expressed by the automobile density, uniform traffic distribution and traffic

* This discussion (of the paper by William J. Cox, Jun. Am. Soc. C. E., published in April 1927 *Proceedings*, and presented at the meeting of May 4, 1927,) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

‡ Received by the Secretary, April 27, 1927.

§ *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 527.

control have added to the carrying capacity of streets, the safety of pedestrians, and the efficiency of the police and fire departments. Without traffic control, modern street movement would be impossible. Since, however, the effectiveness of traffic control decreases with increasing congestion, there is a point beyond which it fails.

As the solution of the traffic problem, traffic control has demonstrated its inadequacy; and traffic regulation, as made possible by physical improvements, is necessary to afford permanent relief. The traffic problem, therefore, is being transferred from the domain of municipal administration to the field of municipal design and construction. In other words, it is becoming the problem of the city planner. Whereas, in the past, its solution has been sought in the control of the traffic stream, in the future, it will be secured by improving the container. Temporary makeshifts will not answer; a radical treatment is required. This is to be found in street widening and in the general improvement of the street plan; nor in these alone, because they are piecemeal methods and, therefore, will not fully satisfy. The traffic problem (including the problem of hazards) affects, and is affected by, all the elements of the city plan. Therefore, it should form a part of a comprehensive plan in which all the physical elements of the city are treated in their mutual relationships.

The conclusions drawn by the author might be enlarged in scope to include practically the whole subject of city and regional planning. The doctrines of centralization and decentralization might be discussed from this new viewpoint, from which also the merits of a well-proportioned radial and circumferential street plan, of a simplified railroad plan with grade crossings eliminated, and of parks and playgrounds as substitutes for the high-hazard residential and tenement areas, might be presented in a new light.

M. O. ELDRIDGE,* ASSOC. M. AM. SOC. C. E. (by letter).†—In his paper, Mr. Cox shows that the automobile public liability insurance rates are lower in some cities than in others. On the ordinary $5 000 and $10 000 policy, the rates for 1927 are, as follows:

New York, N. Y.	$107	Detroit, Mich.	$23
Buffalo, N. Y.	57	Atlanta, Ga.	26
Philadelphia, Pa.	59	Washington, D. C.	20
Providence, R. I.	45	New Orleans, La.	20
Cleveland, Ohio	40	Los Angeles, Calif.	20
Chicago, Ill.	32	Indianapolis, Ind.	18

The rate in Milwaukee, Wis., which has about the same population as Washington, D. C., is $29.

In discussing the relations between actual and calculated hazards, Mr. Cox shows that there are discrepancies between such hazards ranging from 24% of the actual hazards for Portland, Ore., to 109% for Washington. Noting that for Washington the actual hazard is less than one-half the calculated, he attributes this, to a large extent, to superior planning of the city.

* Director of Traffic, Washington, D. C.

† Received by the Secretary, April 28, 1927.

While the avenues in Washington, of which there is one for each State, are wide, the paved portions of the numbered streets running north and south, and the lettered streets running east and west, are narrow when compared with other cities. The average width of the pavement on these streets is from 30 to 32 ft., leaving barely sufficient space for two lines of traffic when automobiles are parked at the curb.

Many other streets in Washington are so narrow as to leave only one traffic lane between parked automobiles. The writer, therefore, cannot agree that these streets are well designed to carry the traffic.

The wide avenues radiating from the Capitol and White House and numerous circles at an angle of 45° to the narrow lettered and numbered streets only tend to complicate this situation from a traffic standpoint. In fact, it is said that this system of street layout was designed by L'Enfant more as a means of internal defense than as a method of handling traffic. As many as ten streets and avenues converge into some of the larger circles in Washington, and this tends to increase the traffic hazards.

During the past two or three years the City of Washington has made considerable progress in reducing the number of traffic fatalities and accidents. The fatality and accident records for the District of Columbia, which is co-extensive with Washington, are as follows:

Year.	Traffic Fatalities.	Year.	Traffic Accidents.
1923	91	1923	8 491
1924	91	1924	9 131
1925	84	1925	9 331
1926	76	1926	6 485

These reductions have been made in spite of the fact that the population has increased considerably during this period and that the registration of automobiles and the number of new drivers has practically doubled.

It cannot be said, therefore, that this improvement has been brought about by the city planners, by the favorable street layout, or by the width of streets, etc. In reality, these reductions are attributable to the following causes:

1.—The establishment of a well laid-out system of boulevard highways at which motorists are required to stop before entering or crossing, thus speeding up traffic on the boulevard highways and making the intersections with adjacent streets much safer.

2.—The elimination of parking on the flow sides of boulevard highways during the morning and afternoon rush hours. This not only facilitates the movement of traffic, but adds an additional traffic lane during the rush hours and reduces the possibility of accidents to pedestrians who may walk out into rapidly moving traffic from between parked automobiles.

3.—The establishment of traffic control on some of the main thoroughfares by means of electric traffic signal lights which operate from 7:00 A. M. until midnight, under a plan of co-ordinated control developed in Washington.

4.—The establishment of a uniform speed limit throughout the District of 22 miles per hour, except on certain sections of boulevard and other high-

ways in the outlying districts, where a speed of 30 miles an hour is permitted as indicated by signs.

5.—The elimination of bad drivers by the cancellation of permits for first offense in leaving after colliding or in driving while intoxicated, for second offense in reckless driving, and for third offense in speeding. Permits are also suspended or canceled for other causes, such as mental and physical defects of drivers. All new drivers are examined on the regulations and are required to demonstrate their ability to operate a motor vehicle safely in congested traffic. An eyesight test is also required.

6.—The adoption of a modern traffic code based on the principles laid down by the Hoover Conference on Street and Highway Safety. This code is being enforced without "fear or favor" by the Police Department and by a Traffic Court which holds sessions every week day, including holidays, from 9:00 A. M. until 11:00 P. M.

C. W. STARK,* ASSOC. M. AM. SOC. C. E. (by letter).†—The author has made a very valuable and unusual contribution to Society publications. It is to be hoped that such discussions will not be so unusual in the future, for the best efforts of the engineer are needed to help solve the traffic problem, to reduce the annual accident toll, the tremendous economic waste due to inadequacy of street and highway facilities, and the unscientific use of existing facilities.

From the viewpoint of the insurance companies and also of the individual motorist it is reassuring to know, as Mr. Cox has demonstrated, that the hazard of the individual automobile does not increase with the number of automobiles. From the viewpoint of the man on the street this is not so cheering, because it confirms his belief that the hazard to himself, be he afoot or in a motor car, is the sum of the hazards to himself from each motor vehicle and, therefore, increases with the number of motor vehicles. Since this number is constantly and rapidly increasing, it behooves each one to bestir himself in every way to reduce the unit hazards for self-protection.

Although Mr. Cox proves mathematically that $w = a$ and, therefore, that the hazard to the pedestrian is the same as that to the occupant of the other motor car, those who do much walking will still feel that there must be some flaw in the reasoning. Perhaps the seeming discrepancy between formula and experience can largely be explained by the author's statement that in a collision between an automobile and a pedestrian, if contributory negligence on the part of the pedestrian is shown, the pedestrian is likely to receive little or no indemnification for his injuries. Since the pedestrian is alleged to be at fault in about one-half the cases, evidently this half does not enter into the formula. Of course, many automobile collisions also do not enter.

It might be added that injuries to persons are seldom averted by such persons leaping to safety from automobiles about to suffer collision, whereas pedestrians are constantly avoiding accidents by jumping and dodging.

* Chf., Highways and Waterways Bureaus, Transportation and Communication Dept., Chamber of Commerce of the United States, Washington, D. C.

† Received by the Secretary, May 2, 1927.

Only by their own agility, exercised in many cases where they clearly have the right of way, do they avert many accidents, and thereby greatly reduce h_p.

Mr. Cox quotes several agencies to derive an average conclusion that pedestrians are at fault in about one-half the pedestrian accidents. Obviously, it is much easier to get testimony from a living motorist than from a dead pedestrian. Obviously, also, if a pedestrian is found injured or dead in a roadway in the middle of a block, it is reasonably certain that he was not crossing the street on a cross-walk. But there may be no witnesses or other proof of the fact that the motor vehicle was going at double the legal speed, or was on the left-hand side; that the pedestrian saw the vehicle and would have had ample time to get out of its way, but that the driver wilfully or heedlessly ran him down. On the other hand, if the pedestrian is hit on a cross-walk there is always the chance to prove that he wavered or darted forward suddenly, or did something else that the oncoming motorist could not foresee and, therefore, he was at fault. Pedestrians jumping for their lives from the paths of reckless, unidentified drivers, have darted into the paths of law-abiding drivers. Cause of accident—confusion of pedestrian.

Thinking further along these lines, those who live and walk in Washington, D. C., will not be lulled into a false sense of security by the fact that the "actual" hazard is less than half the calculated hazard, because there comes into the problem another element, which could not very well be calculated by Mr. Cox, namely, the indifference of the Washington police and apparently the prosecuting authorities to the welfare of the pedestrian. The great majority of Washington traffic officers operate their signals as if pedestrians were non-existent, shifting traffic without regard to persons on foot marooned in the middle of the street, giving no interval for the clearing of intersections, and permitting vehicles to crowd and obstruct the cross-walks and make illegal right turns with little interference. Naturally, such officers are of little help in proving the responsibility of motorists when pedestrians are injured. The writer's personal experience has been that in Washington the interest of the public authorities in the welfare of the pedestrian approaches zero. For this reason the likelihood of insurance companies having to pay claims resulting from pedestrian accidents approaches zero, and, therefore, h_p approaches zero.

The wide streets of Washington certainly provide an opportunity for safety not enjoyed by many cities. They make it possible for vehicles to operate with a minimum of interference with each other; but they also make possible higher speeds and more rapid rounding of corners, more lanes of traffic and greater distances for the pedestrian to negotiate, creating additional hazards for the pedestrian and calling for special effort on the part of the regulatory authorities to overcome those hazards. Because such special effort is not in evidence, the writer has heard more than one man, coming from the hurly burly of New York City, with its abnormally high value of H, to work with committees of the National Conference on Street and Highway Safety, express fear of walking on Washington's streets, and a desire to get back quickly to the comparative safety of the metropolis.

From the author's opinion that improved methods of handling traffic at street intersections will have little effect on collisions between automobiles and pedestrians, the writer emphatically dissents. Is there not a very positive relationship between the treatment accorded the pedestrian at intersections and his general attitude toward traffic? If he has no guaranty of protection when he proceeds on a "Go" sign—if he has to wait intolerably long periods for the signal to go, because there is little wheel traffic going his way, and when he does get the signal has to detour around cars stopped on the cross-walk and dodge other cars making all manner of turns, and then, perhaps, be left in the middle of the swirl by a sudden shifting of the signals—and if at unofficered intersections he has to weave his way through solid lines of vehicles that completely obliterate the cross-walks, and have the motorists blow their horns and crowd up to close the gaps so he cannot get through—in short, if the whole scheme of traffic control makes him a mere incident, he is, of course, going to cast rules and regulations to the winds and cross the streets whenever, wherever, and however his judgment, faulty and inconsiderate though it be, determines. Give him a decent chance at the cross-walks. Mark them conspicuously and keep standing motor vehicles off them. Instil in motorists a wholesome respect for these intersections of sidewalk and roadway, and show pedestrians that regulation of pedestrians does not mean merely restriction for the convenience of motorists—and see whether this does not bring about changes in walking and in motoring habits that will cut pedestrian fatalities in half.

This dissertation on the pedestrian may seem a far cry from engineering, and unscientific. Surely, however, the engineering mind must see that, with pedestrian fatalities constituting about two-thirds of all fatalities in which the motor vehicle is involved, the pedestrian problem is the most serious phase of the traffic problem; and it is about the most neglected.

Mr. Cox's figure of $15 000 000 annual saving to the State of Massachusetts (after cost of administering the laws is deducted), because of its advanced motor-vehicle laws, should be brought to the attention of all those who believe that the cost of administering such laws would be prohibitive.

Reasonable, enforceable rules of the road are one of the essentials of advanced motor-vehicle laws. Another essential on which Mr. Cox might well have laid more emphasis is the licensing of drivers. All four of the States which he cites as having advanced legislation in 1922 require and did require in 1922 that all operators and chauffeurs be licensed; and it is important to note that in none of these States is this licensing a mere perfunctory registration of drivers. The applicant for a license must prove by an examination that he has the qualifications for a safe driver. Such examinations permanently eliminate a small percentage of hopeless incompetents and compel a much larger number of applicants to familiarize themselves with the laws and regulations and really to learn to drive safely before they are turned loose on the highways. This system costs the State some money, but very little in comparison with such savings as Mr. Cox's figures show. No State having tried the licensing and examination system has discarded it.

The skeptics should not overlook the significant statement in the paper that "good traffic regulation is usually the outgrowth of high hazard—of conditions so intolerable that they force improvement." Experience is a costly school, but perhaps the only possible school for the dozen States that have suffered most and gone farthest in working out sound legislation. While these States are grappling with the problems still unsolved—and the relation of pedestrian and motorist is one of the gravest of these—the remaining three dozen States have an opportunity to profit by the experience of the dozen.

Engineers do not need to be told the advantages of intelligent zoning and traffic segregation or of well-planned street layouts. What they do need to realize is that the engineering problems are being handled to a large extent by others than engineers, and that they should hasten to take over that part of the traffic problem for which they are peculiarly fitted, so that mistakes already made can be corrected as promptly as possible, and new and costly mistakes can be avoided.

L. G. Holleran,* M. Am. Soc. C. E.—This paper is both timely and interesting. Its only defect is that it does not go far enough. It gives an outlook on this problem from a new angle and arrives at a solution in a novel way; but it is regrettable that more ground could not be covered in the study of methods for reducing the automobile hazard.

This hazard is only one phase, although a most important one, of the large problem of taking care of automobile traffic with the inadequate street systems and facilities which exist in practically all American cities. As some one has aptly put it "we are building 40-story cities around street systems designed for 3-story towns"; he might have added "and we are suffering the consequences".

It is enlightening to examine Mr. Cox's formula and to see what application it has to existing conditions. He found that this formula, $H \propto \frac{P}{M}$, did not hold good for a number of cities and came to the conclusion that several factors might serve to modify it. These factors will be discussed later.

On considering the formula, it is apparent that the hazard can be reduced in two ways, first, by decreasing the population, P, and, second, by increasing the street mileage, M. It is obvious that the population is decreasing in few cities of the United States, and, in fact, is increasing in most of them, so there is no hope for immediate relief in that direction. Accepting the formula, therefore, the only manner in which relief is possible is by increasing the mileage of streets and roads in such a way as to better distribute automobile traffic throughout the area under consideration.

That is the method by which the Westchester County Park Commission is attacking the problem. The County has an area of about 450 sq. miles and a population of about 500 000. The County is in reality an urban district subject to the limitations inferred in the paper. It is the front dooryard of the City of New York; in addition to traffic originating in the County, its

* Deputy Chf. Engr., Westchester County Park Comm., Bronxville, N. Y.

roads and streets must carry a tremendous through traffic going to or coming from the city. This through traffic has been carried almost wholly on two routes: The Albany Post Road and the Boston Post Road. Each of these routes leads through the business center of every city and village it traverses, thus greatly aggravating local traffic conditions and increasing the automobile hazard until a point was reached a few years ago where it was realized that remedies must be found or the development of the County would be seriously retarded.

The result was the appointment of the Commission which, after investigation, came to the conclusion that the only practicable way to relieve conditions was to lay out new routes, parallel to the over-taxed existing routes, to be located away from the business centers of the cities and villages, and as far as possible in undeveloped territory for reasons of economy as well as to provide for a better distribution of the population. Of such new routes 140 miles have been laid out, and the Board of Supervisors has appropriated nearly $40 000 000 for the acquisition of the real estate and the construction of these new routes, in addition to the land and construction work necessary for the development of recreational facilities throughout the County.

In order to facilitate the passage of traffic, as well as to reduce the automobile hazard, the new driveways are being constructed with a pavement width of from 40 to 80 ft. Truck traffic will be segregated on driveways other than those carrying passenger traffic. Grade crossings at all the principal street intersections are being eliminated, curves super-elevated, non-skid pavement used, modern lighting installed, and traffic regulated in accordance with approved methods.

Working along similar lines, the State of New Jersey is building a 50-ft. driveway, 13 miles long, from the westerly end of the Holland Tunnel to a point near Elizabeth, at an estimated cost of more than $30 000 000. New York City is to build an elevated roadway on its west side from the easterly end of the Holland Tunnel to connect with the southerly end of the Riverside Drive at 72d Street. The City of Chicago, Ill., has recently widened Western Avenue from 60 to 100 ft., for a distance of more than 20 miles, and is cutting through additional streets at great expense. Detroit, Mich., is building boulevards, 204 ft. or more in width, and Wayne County is carrying the boulevards from the city line for many miles out into the country. Philadelphia cut a wide diagonal street leading out from the heart of the city for a distance of about 1¼ miles a few years ago. Other cities are attempting to solve the problem by similar methods.

Such a program for Westchester County is possible largely because the County happens to be a very wealthy community. It, also, fortunately had within its borders leaders and officials possessed of those qualities of imagination and foresight which insured support of the comprehensive program proposed by the Commission. If men of vision and courage could be found to lead, most communities could discover the way to finance the program of improvements in its street system necessary to remedy unsafe, uneconomical, and unsatisfactory conditions.

If the losses due to fatal and non-fatal accidents, loss of business, loss of the time of drivers of private and commercial cars, and the general inconvenience of traffic congestion, could be evaluated and credit allowed for the increase in real estate values resulting from the improvement, it would, in most if not all cases, be found that the actual return in dollars and cents more than justified the expenditures necessary to attack the problem in the comprehensive way it deserves. It is a fact, however, that due to lack of proper leadership and other causes, a great many communities will decide that they cannot finance the large costs required to effect a relatively complete cure. It is necessary, therefore, to consider what means can be found to alleviate conditions with the funds which can usually be made available.

Mr. Cox concludes that the automobile hazard may be reduced in four general ways. These methods are of equal effect in solving the problem of automobile traffic as a whole. In addition, there are a number of others which, in the absence of funds to attack the problem in a comprehensive way, will tend to better distribute the traffic; facilitate the passage of automobiles through the streets; and, at the same time, reduce the hazard:

1.—Uniform traffic regulations throughout the United States. As it is now, even adjacent municipalities have different regulations which lead to the confusion of strangers who often constitute a relatively large proportion of the total number of drivers on through traffic routes.

2.—Widening of existing streets parallel to those for which the records show the greatest hazard, with this end in view, to attract a part of the traffic away from dangerous localities. One additional lane of traffic can often be secured by narrowing the sidewalks. Where this is insufficient or impracticable, streets can often be selected for widening where the damages to existing structures are at a minimum. It is of little avail, however, to widen streets in the congested district, as such widening merely serves to attract traffic from adjacent parellel streets and the congestion soon becomes worse instead of better.

3.—Connecting up segments of existing streets to form new thoroughfares, where, due to improper control of real estate layouts, or other reasons, the streets in adjacent developments do not connect. Increased mileage of through streets and better distribution of traffic may often be secured in this way at comparatively small cost.

4.—Re-routing of trolley cars from congested streets to less used parallel streets. A few years ago when Chicago had a strike of street railroad employees and the surface cars ceased to operate for several days, traffic counts at a number of street intersections made during the strike and compared with counts taken afterward showed that the capacity of the streets decreased about 50% when the trolley cars were in operation.

5.—Paving through streets in the outlying districts to a width of not less than 30 ft. to provide a third lane of traffic so that one automobile may pass another in safety. Statistics show that a considerable proportion of automobile accidents occur while one car is trying to pass another.

6.—Providing sidewalks or paths along streets and roads in the outlying sections and along through traffic routes.

7.—Preventing the parking of automobiles in congested districts, leaving the streets free for the passage of traffic.

8.—Restricting the number of taxicabs and forbidding the practice of "cruising" for business.

9.—Removal of obstructions and encroachments, both public and private, from sidewalks and streets wherever possible.

10.—Preventing the practice of storing building materials and conducting building operations on parts of streets or sidewalks.

11.—Preventing the use of the sidewalk and a part of the street for considerable periods of time for the purpose of loading and unloading merchandise. This practice often forces pedestrians to make use of the streets and increases the possibility of accidents.

12.—Reducing the number of street openings for sewers, water, gas, or other public utilities, to a minimum, and making it a provision of the permit that the work of making and closing these openings shall be done in the shortest possible time. The repair of the pavement should always be done by the municipality at the expense of the person to whom the permit is granted.

13.—The use of the proper types of pavement for different traffic conditions.

14.—The prompt and efficient repair of the street surface when it begins to deteriorate.

15.—Increasing the radii of curbs at street intersections, not to invite an increase of speed in turning the corners, but to prevent cars with a long wheel base from encroaching on the wrong side of the street at turns and thus increasing the possibility of collisions with other cars as well as with pedestrians.

16.—Where it is impracticable to eliminate grade crossings at important intersecting streets, it is often possible to locate a circular island at the center of the intersection, requiring a rotary movement of traffic. This is relatively ineffective, however, unless the island has a rather large diameter, probably not less than 60 ft.

17.—Loading platforms raised about 6 in. above the street have been provided in some cities for the safety of passengers boarding or alighting from trolley cars. Safety isles for the same purposes have also been provided by marking off parts of the pavement with temporary posts connected with rope.

18.—The establishment of more playgrounds in or near the congested districts for the purpose of keeping children off the streets.

19.—Last, but not least, educating the public as to safety measures, beginning with the children in the schools.

It must be admitted that many of these suggestions are palliatives and not cures, but they will continue to be necessary until the street systems can be made over and a rational and uniform method of procedure for handling automobile traffic can be worked out and put in operation.

S. JOHANNESSON,* M. AM. SOC. C. E.—In his excellent paper the author comes to the conclusion that generally the personal injury hazard of automobile operation is proportional to the population density. It appears, however, that other factors may be involved, as may be illustrated by the following example. Traffic counts have been made by the State Highway Department of New Jersey at Rahway on July 4 every year since 1921, except 1925. The counts have been as follows:

Year.	Number of Vehicles.	Year	Number of Vehicles.
1921	12 153	1924	26 573
1922	16 610	1925	....
1923	18 314	1926	43 735

* Designing Engr., State Highway Comm., Jersey City, N. J.

These counts indicate an increase of traffic during the 5-year period of 259 per cent. The exact population increase during the same period is not available, but it is probably not more than 20 per cent.

In this case the traffic increase is exaggerated by the traffic in transit from New York to the New Jersey coast resorts; nevertheless, it is a fact that the traveling habit is steadily on the increase. Not only does the ratio of number of cars to total population increase, but the average distance each car travels is increasing. The rate of increase of the traveling habit may decrease when the congestion of traffic on existing streets becomes excessive; but whenever new roads are built or present roads are improved the traveling habit will increase.

Consequently, it would seem as if the traffic density (which presumably is proportional to the personal injury hazard) cannot be taken to be proportional to the population density, unless the comparison is made at a definite time. It is possible to imagine, for example, a community in which the expression, $\frac{R}{M}$, might be the same in 1921 and 1926; but it seems probable that the personal injury hazard in 1926 would have increased materially over that in 1921 on account of the materially greater density of traffic.

In his "Conclusions",* the author suggests traffic segregation at intersections as a means of reducing the hazard. At present, the New Jersey State Highway Commission is constructing a new highway leading from the Holland Tunnel Plaza in Jersey City through Newark and Elizabeth, where connections are made to existing traffic routes leading west and to the New Jersey coast resorts. On this highway, "space segregation" of traffic is being provided by building it so that there will be no grade crossings with either railroads or existing streets. This space segregation is being provided largely on account of economic considerations. Investigations have been made which indicate that on this road, which is expected to carry about 18 000 000 vehicles per year, the cost of time segregation, or in other words, the loss in cost of operation of the vehicles on account of the delays at crossings, would exceed the additional cost of construction on account of the space segregation.

The results of these investigations indicate that, where the traffic is heavy, it may be worth while to consider the comparative economics of time segregation and space segregation.

W. Graham Cole,† Esq.—The author applies a strict mathematical formula to a problem that involves a consideration of engineering principles and human psychology. These principles are so inter-related that only an incomplete and inaccurate picture of the problem is obtainable from a purely mathematical viewpoint. Thus, if the formula, $h = \frac{P}{M}$, is carried to the nth degree of reasoning, namely, if M is sufficiently large in comparison with P, the value of h will approach zero as a limit.

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 537.

† Safety Engr., Metropolitan Life Insurance Co., New York, N. Y.

It is obvious, however, that if pedestrians can be eliminated entirely from thoroughfares used by vehicular traffic no accidents between the two will be possible. Before this limit is reached a zone will be found in which the intermittent and unexpected appearance of a pedestrian on a thoroughfare, otherwise frequented by vehicles, will produce a greater hazard to the individual pedestrian than he now encounters at heavily congested intersections, where his presence is anticipated by the motorist. This is borne out by the experience of many cities, which shows that the majority of traffic accidents do not occur in congested sections, but rather in outlying sections, where lack of congestion permits of speeding and reckless practice.

The theoretical solution of the pedestrian-motor accident problem is the complete and permanent segregation of these two types of street users. Such steps, however, would not affect, to any great degree, the problem of vehicular collisions with each other or with stationary obstructions.

The segregation of vehicular traffic by direction so as to make impossible the intersection of one line of moving vehicles with another would prevent the intersection accident, but would leave unaffected many other types. Such segregations are impractical from a cost standpoint, if from no other, except at a few unusual locations in the most congested communities. As a general solution to the traffic problem, they are beyond the realm of practical consideration. The traffic problem is a serious one, dealing with the deaths of about 24 000 persons per year, a number that is increasing at an approximate rate of 1 death per 1 000 added motor vehicles. On account of this increasing toll, methods of dealing with the problem that are within the scope of practical application by the average community are not only highly desirable, but absolutely essential to the public welfare.

The Engineering Profession can render a great service to humanity by giving to this problem the same practical consideration and study that it has given to other problems. It can assist the Nation, the State, and the individual community to devise and take definite steps leading to the correction of existing conditions.

Mr. Cox has pointed out that the actual hazard, as he has developed it, is greater than the calculated hazard in many cities where traffic control has been practiced. Granting that the basis of his calculations is correct, this fact indicates that traffic control has been applied in the places most needing it; but it may also indicate that the most effective methods of traffic control have not been devised. The fact does not condemn the practice of traffic control, but rather emphasizes the need for greater and more intensive study of this important branch of engineering.

Accidents are largely the result of the inability of the human race to modify its habits of life quickly to conform with the tremendous changes of environment brought about by engineering advancement and commercial development. The average pedestrian still wishes to amble across a busy thoroughfare carrying thousands of powerful machines, in much the same way that his ancestors sauntered across a village street between a few slow-moving, horse-drawn vehicles. Engineering devices can be developed to alter certain

environments so that they will conform more closely with the unchanged habits of the public. On the other hand, educational methods must be applied, in order to alter habits to satisfy changed conditions.

Accident prevention work in industry has demonstrated that only a small reduction of accidents can be effected by mechanical guards and engineering revision. This work must be supplemented by definite and organized educational activities. Likewise in the matter of public accidents only a small reduction will be made if the educational phase of the problem is overlooked. Through this educational work the individual citizen must be reached, his interest appealed to, and he, as an integral part of the community, must be taught new responsibilities.

In a number of communities, organizations have been perfected for the purpose of conducting continuous safety educational activities. The gratifying results obtained have proven beyond doubt that a community can be organized for the prevention of accidents, very much as industrial plants have been organized.

Of the list of cities given on Fig. 3,* nine have been conducting organized educational work for some time. Of these nine, seven have an actual hazard below the calculated hazard.

HAROLD M. LEWIS,† M. AM. SOC. C. E.—The author has presented a strong case for the formula he has set up—that automobile accident hazard within a city is directly proportional to the population and inversely proportional to the street mileage. Such a formula implies that the most practical way of reducing such hazards is by decreasing the density of population along a typical street within a community. This points directly to some form of decentralization that will bring about a better distribution of population and activities within cities. The need for such a re-arrangement has been much discussed in recent years, and it seems desirable for many reasons.

Mr. Cox assumes that motor-vehicle density is not the controlling factor, because he has developed from other factors an equation which fits his accident statistics. This is not proof. It is quite possible that by starting with motor-vehicle densities and applying necessary corrections, such as he has applied to street mileages, automobile density may also be proved to be an important controlling factor. Although fatality records vary too greatly in small areas to yield any general principles, they should indicate actual trends in large areas, such as States. They do have advantage in being more complete than accident records.

The National Conference on Street and Highway Safety presented very complete fatality statistics for 1923, 1924, and 1925. These were tabulated for total State areas and also for the total rural districts and total cities within each State of the United States. An analysis of the 1923 statistics showed that there was a larger variation in the fatality rates in the rural districts than in the cities, as might be expected because of the greater variation in conditions. The maximum rate in the groups of rural districts was

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 527.

† Executive Engr., Regional Plan of New York and Its Environs, New York, N. Y.

38.6 per 100 000 of population in California, and was greater than the rates for any of the various groups of cities. The average of the rural district rates, however, was only 11.1 fatalities per 100 000, and the fatalities in the total rural districts listed amounted to only 10.4 per 100 000. In contrast with these figures, the statistics for fatalities in the different groups of cities showed an average rate of about 20 fatalities per 100 000, or almost twice as great. Data for the later years showed approximately the same conditions.

One of the most striking features in the tables was the small variation for the different groups of cities, which indicated that the motor-vehicle hazard in any place was primarily a result of urban conditions and quite independent of the size of the city. Comparing the fatality rates of the different States with the automobile density in terms of cars per capita, it appeared that communities with the least number of cars per capita had the lowest fatality rates and *vice versa*. California, which had by far the greatest number of vehicles per capita, had the largest fatality rate. The States with more or less average fatality rates showed no such direct relation, and other factors apparently obtained.

It is hardly enough to know that one must have better motor-vehicle legislation, or a street system like Washington, D. C., in order to reduce motor-vehicle hazard. Other cities cannot afford to duplicate Washington in its rather extravagant provision of street areas. Proper planning must be based on analysis of the more specific causes of accidents, such as has been made by the National Conference on Street and Highway Safety, and by Connecticut, Wisconsin, and other States. The Connecticut analysis indicates that the ratio of accidents to registration increased fourfold from 1918 to 1925, while the automobile density increased only two and one-half times. These data must be discounted to a certain extent to allow for more complete returns in the later years, due to improved methods of reporting accidents.

TABLE 5.—Automobile Facilities in New York and London.

Year.	New York City.	London Metropolitan District.
1917	441	471
1918	503	440
1919	696	533
1920	730	512
1921	851	460
1922	846	560
1923	934	569
1924	936	720
1925	1 002	742
1926	1 023	895

On the other hand, the statistics of the National Conference on Street and Highway Safety show that the ratio of fatalities to registration decreased steadily from 1917 to 1924. The same trend was indicated by an analysis of fatality figures for New York City, New York State, and the entire United States, from 1917 to 1923, made by the Regional Plan of New York and Its Environs. It appeared that, in general, the number of fatalities in each of

these widely different areas had been increasing as the three-quarters power of the motor-vehicle registration. During the four years ending with 1926, there was a decided change in this relationship in New York City when motor-vehicle fatalities increased as only about the one-third power of the registration.

It may be of interest to compare motor-vehicle fatality figures for New York City with those for the London Metropolitan District. New York City has less than one-half the area of the London Metropolitan District, but has an automobile density three or four times as great. Table 5 gives the information from 1917 to 1926.

It is estimated that in the entire New York Region—defined as all the area within approximately 50 miles of the City Hall—there were about 1 870 motor-vehicle fatalities during 1925. This enormous loss of life indicates very forcibly the seriousness of present conditions. The London figures show that the problem also exists in the large metropolitan areas in other countries.

J. K. FINCH,* ASSOC. M. AM. SOC. C. E.—The author has taken up a problem of the first magnitude—think of New York City alone spending $75 000 000 annually on automobile accidents. He has pointed out clearly certain laws of automobile hazard—that these laws must be taken into account in city planning if the hazard is to be kept down to a minimum. Unfortunately, the problem is generally the reverse, namely, to reduce the hazard in cities, in the planning and growth of which no attention has been given to this Twentieth Century problem.

About ten years ago, under Police Commissioner Woods, the speaker served on a special committee of three to study the street accident problem of New York. Fairly reliable data had been secured by the Police Department which showed about 22 000 street accidents of all kinds in 1916. More than 25% of these accidents were due to "collisions" between pedestrians and automobiles, and, therefore, it was clearly indicated that the problem, already alarming, would continue and grow with the rapid increase in automobile use. In 1926 there were more than 50 000 street accidents in New York.

Unfortunately, little time was available for this study, as the Committee was appointed just at the close of Mr. Woods' administration. Such studies as were made, however, established two fundamental laws of street accidents. The principal type of accident being due to a collision between a pedestrian and an automobile, these laws have to do first with the population characteristics which affect the accident rate; and, second, with the traffic characteristics, which tend to increase the number of accidents.

The first of these laws was the relation of population density to the number of accidents, which has been so clearly brought out by Mr. Cox's studies of insurance rates in cities of different population densities. He has also suggested a unit, population per mile of street, that reduces this hazard element to comparable figures. This unit applies equally well to entire cities or to parts of cities. For example, the Committee went so far as to compare a

* Associate Prof., Civ. Eng., and Director, Summer Session, Camp Columbia, Columbia Univ., New York, N. Y.

map showing the location of street accidents in New York with one showing the density of population. The coincidence was striking. The reason for this law appeared to be simple—the great bulk of the accidents was due to collisions of vehicles and pedestrians. This means, in general, the greater the number of pedestrians, the more accidents.

In continuing this line of thought, however, and arguing the more vehicles, the more accidents, the conclusion fails. Statistics showed that the maximum number of accidents did not occur, for example, on Fifth Avenue where there was a constant and almost solid stream of vehicles and where the sidewalks were crowded with people. On the contrary, they proved that the second law, which applies to vehicles, was different, that is, the number of street accidents was greatest on streets used by a variety of vehicles, such as trucks, passenger automobiles, and trolley cars, and where there was not sufficient congestion to prevent a fairly high speed with open spaces between vehicles. Eighth and Tenth Avenues below 59th Street, for example, and First Avenue throughout its length, are streets of this type with especially high accident rates.

One fundamental law in reference to city planning or to the design of new traffic lines is clear from this, namely, that new traffic routes, such as new streets, or bridge or tunnel outlets, should be kept away from centers of high population density. Thus, in attacking the street accident problem as it now exists in any city it is important to diffuse population and to concentrate traffic.

The chances of diffusing the population, or, to be more exact, of reducing the population density in heavy traffic sections, seem very small. Mr. Cox has mentioned zoning laws. It is also clear that the development of low-priced homes in suburban sections with adequate rapid transit facilities, may aid in this direction. In general, however, the population densities of great cities is increasing and is certain to continue to increase with the construction of the skyscraper apartments, offices, and hotels. It seems clear that little can be done through attempting to secure a diffusion of population.

This narrows the principal line of attack to the traffic problem. Three methods are available for securing a concentration rather than a diffusion of traffic. What is needed is to secure an orderly stream of traffic moving at as high a speed as is reasonable and possible. Fifth Avenue, with trucking eliminated and traffic controlled by semaphores, is probably, in spite of its pedestrian density, one of the safest streets in the world. Three methods have been shown to be effective in securing the desired results—the semaphore system, traffic segregation, and the one-way street. It has also been shown, however, that these plans cannot be applied to a few streets, for this simply results in deflecting traffic to other streets—often in diffusing it through areas of high population density, which has just the reverse of the desired effect.

A map of New York City shows how completely these laws have been neglected in planning traffic routes and gives ample reasons for the high automobile hazard. There have been, of course, sound reasons based on topography, economy, and transportation needs, for most of these traffic routes, but the problem of the automobile hazard introduced by them apparently has

received little consideration. Indeed, the usual traffic idea is to break up the solid streams of traffic which enter the city over bridges and through streets and to distribute it among the side streets and avenues—a plan directly opposed to a fundamental principle of hazard reduction. Furthermore, the close regulation of traffic on certain avenues has the same effect—that of forcing traffic into other streets and avenues, thus increasing the hazard.

The lower East Side bridge section of the city is the section of highest hazard. Here, there is a dense population with several main traffic routes, trucks, trolley cars, and passenger automobiles, feeding toward the bridge entrances. Obviously, wide avenues with traffic segregated is the only solution of this problem. Costly approaches and street widening are as essential as bridges and tunnels.

The automobile hazard in Manhattan cannot be reduced materially except through the adoption of a comprehensive traffic plan for the entire Island, a plan based not only on traffic requirements—capacity and speed—but also on the laws of traffic hazard. Isolated efforts here and there over the streets and avenues of New York will have comparatively little effect; indeed they may increase the hazard. A comprehensive plan of traffic segregation, one-way streets and avenues, and semaphore operation for the city, as a whole, is essential to facilitate traffic and reduce the hazard. All future street and bridge plans should be studied from the standpoint of hazard as well as of traffic facility.

G. G. KELCEY,* ASSOC. M. AM. SOC. C. E.—This paper develops a formula by which traffic accident hazard may be determined. Some factors which might be expected to have an influence in determining hazard are eliminated on the assumption that their bearing is slight and that they will, under average conditions, balance each other. Therefore, the question concerns the accuracy as to the premises and in the development of the conclusions.

A summary of factors which may be considered as having some bearing on the automobile accident hazard should include the following:†

Motor Vehicle Group.—

Motor vehicles per person.
Motor vehicles per square mile.
Motor vehicles per mile of surfaced road or street.
Average yearly motor vehicle mileage.
Distribution of automobiles by type, as trucks, taxicabs, etc.
Concentration of majority of vehicles over limited number of routes as distinguished from a general distribution of vehicles over many streets.

Population Group.—

Persons per motor vehicle.
Persons per square mile.
Character and intelligence of population.
Driving experience and ability of vehicle operators.

---

* Mgr., Traffic Eng. Div., Am. Gas Accumulator Co., Elizabeth, N. J.

† From data comp. by Hawley S. Simpson, Jun. Am. Soc. C. E., Traffic Engineer, Detroit, Mich.

Miscellaneous Factors.—

Playground provision for children.
Simplicity of traffic regulations.
Enforcement of regulations.
Methods used for physical regulation and control of traffic through the use of other traffic aids.
Street design with respect to crown, curb radii at intersections, width of streets, etc.
Average visibility at intersections.
Sidewalks, or the lack of them, in outlying areas.
Topographical conditions.

All these factors have a bearing, and others might be added. Certain of them are fixed conditions and not subject to modification, while others, such as street design, visibility, and driving habits, are subject to treatment and control. It would seem that a formula to determine hazard should be based primarily on the mass factors of population, automobile registrations, and street mileage. The result, however, should be modified by the factors that are subject to a direct control and that vary from one community to another in direct proportion as the treatment of correctable conditions is successfully undertaken.

Although traffic accident data at present available are neither searching nor accurate, there is sufficient information at hand to indicate that 30 to 40% of persons killed and injured by traffic are children. It further indicates that about one-half of all the persons injured or killed are pedestrians "jay-walking" between blocks, and that about two-thirds of these are children. Two conclusions are obvious, if all "jay-walking" could be stopped, presumably 50% of the injuries and deaths could be prevented; if children could be kept from playing in the streets or crossing in the middle of the block, it is reasonable to suppose that a large proportion of that group of injuries and deaths would be prevented. It seems clear, therefore, that any hazard formula would be incomplete if it failed to provide for modification to suit local variables when sufficient control is established over those accident-producing factors that are subject to moderation. In any event, it is believed that hazard will tend to approach, as a limit, the base values which depend on the mass factors of population, registrations, and street mileage, and which may be considered as the inevitable and unavoidable price of progress.

If, as in the paper, "hazard" refers to all class of vehicles and if it may be measured by insurance rates, then such insurance rates must be the average charge for insurance per car for the various hazards insured. This average charge for insurance must then be the total insurance paid by cars of all classes, divided by the number of cars. If this is the case, it does not seem proper to predicate conclusions with respect to hazard in general on insurance experience and rates, for the following reasons:

(1) It is generally understood, although not definitely confirmed, that less than one-half the motor vehicles in an average community are insured and that the percentage varies.

(2) The lower the rates, the greater are the number of insured and the less is the average individual risk.

(3) As the rates are increased, the preferred class of risk drops out until, as in New York City, the rate is so high that there is an undue percentage of bad risks.

(4) The majority of cars insured in cities, such as Indianapolis, Ind., are private, and may be expected to cover less than 10 000 miles per year; whereas the majority of cars insured in a city like New York are commercial and have a mileage range from 25 000 to 100 000 miles per year in the case of many taxicabs. Hence if the degree of hazard is to be established, all vehicles involved must be included in the calculation.

If it is true that higher rates do cause the preferred risk to drop out leaving the costly risk, and so on in a vicious circle, and if many drivers who are greater risks cover many times the average mileage of a lesser risk, is it not reasonable to assume that insurance experience is not an exact measure of a hazard of the city as a whole, but reflects instead an unbalanced situation?

This brings in the matter of automobile club insurance competition in such cities as Chicago, Los Angeles, San Francisco, and Detroit. These clubs have the advantage of a membership fee, and sell their insurance to members at a very low sales cost and, it is assumed, without profit. In all these cities insurance rates are exceedingly low. For example, in Detroit the Automobile Club began selling insurance in March, 1922. Conference rates, to which Mr. Cox refers, continued without change through 1923, but, in 1924, public liability insurance on a Ford dropped 30%, on Buicks, 32%, and on Cadillacs, 31 per cent. The other cities referred to are understood to have had a similar experience.

It seems reasonable to conclude that the insurance companies are subject not only to variations in risk, but that their rates are subject to the same competitive conditions which cause price variations in gasoline. If this is so, insurance rates cannot be taken as an accurate basis for measuring automobile accident hazard.

As a general conclusion, the paper suggests that: "The variation expressing the personal injury hazard * * * as population divided by street mileage, adds a warning against such degrees of congestion of population as prevail in almost all older large cities." Again, "the diffusion of population leads to a decrease in the number of possible collisions per unit distance of travel and thereby reduces hazard". And, finally, "most automobile collisions with pedestrians occur in residential districts." These statements seem to be contradictory since, for the most part, residential districts are areas in which the population is diffused.

Recently, some one asked the head of a large insurance company to state where, in his opinion, was the safest place. His answer was, first, on a railroad train; and, second, at the crossing of 42d Street and Fifth Avenue, New York, when it is carrying a maximum load of traffic.

It seems reasonable to suggest that, under conditions either of congested movements of vehicles or pedestrians, the hazard per unit vehicle or pedestrian is reduced, although the total reported accidents may be increased. In general support of this conclusion, Fig. 4 has been prepared from 1926 figures

furnished through the courtesy of W. A. Van Duzer, M. Am. Soc. C. E., Deputy Engineering Executive of the Pennsylvania State Highway Department. This diagram shows the volume of traffic and the number of accidents for each hour of the day. From this information the degree of hazard (accidents, in terms of traffic) is determined. The curve indicates a rising hazard as the density of traffic decreases. It is reasonable to suppose that the declining density of traffic is, in part, responsible for this increase in hazard.

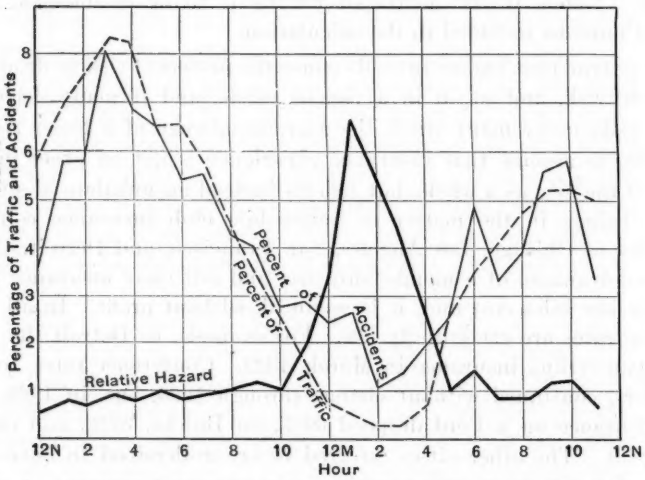


FIG. 4.—ACCIDENT HAZARD BY HOURS, PENNSYLVANIA STATE HIGHWAYS.

Mr. Cox is to be commended on his effort to clarify a subject which is shrouded in the fog of confusion, misinformation, and opinion. It is the speaker's conclusion, however, that he has not included all the factors necessary, in his formula, and that, although insurance rates and experiences are an index to the hazard experiences of selected classes and groups of cars, they are of doubtful value in measuring the hazard of a whole community.

RAYMOND A. O'HARA,* ASSOC. M. AM. SOC. C. E.—That automobile hazard may be reduced by proper planning, is recognized if not appreciated. Roads on which intersecting highway grades have been separated minimize the chance of accident. Good examples of this type of highway are the Bronx River Parkway, extending from the northerly end of Bronx Park in New York City to the Kensico Dam in Westchester County, New York, and the highway being constructed by the New Jersey State Highway Department to connect with the Holland Vehicular Tunnels. The proposed West Side Elevated Roadway in Manhattan is another example.

There is a relationship between building bulk and automobile traffic. Replacing buildings of low heights with higher buildings brings additional traffic into the district. Further congestion and increasing hazards may be expected with increasing building bulk. Traffic control signals have played a part in the reduction of accidents and fatalities.

* Prin. Asst. Engr., Regional Plan of New York and Its Environs, New York, N. Y.

JOHN A. MILLER, JR.,* ASSOC. M. AM. SOC. C. E.—Where comprehensive traffic regulations have been adopted, as Mr. Cox points out, the accident hazard often is worse than in other cities. This suggests that the method of regulation is not always well suited to the needs of the situation. In New York City, for example, an elaborate system of traffic signals has been installed. Unfortunately, however, they are so operated that many of the beneficial results which might easily be obtained, are lost.

Several shortcomings stand out prominently. Signals all flash simultaneously over long stretches of important thoroughfares. Thus, all traffic on these streets starts and stops at regular intervals. There is no speed at which the driver of the vehicle may go and be assured of making continuous progress. Naturally, he tries to cover as much ground as possible while the signal is clear. This leads to excessive speed and many dangerous attempts to "beat" the lights.

In other cities this difficulty has been overcome by the installation of lights operated on a progressive or wave plan. In essence this is nothing more than alternation of the lights in groups of two or more so that a vehicle starting when the light changes to green can proceed at a regular moderate pace throughout the entire area controlled without stopping, successive groups of lights changing to green as the vehicle approaches them. Under this system traffic as a whole moves faster than under the system used in New York, but excessive speed is penalized. From the point of view of reducing the accident hazard, it has marked advantages over the synchronized signal system.

Another glaring defect of the New York traffic signal arrangement is its failure to adjust the time intervals to suit conditions at the individual intersections. Traffic is held up very frequently when there is no cross traffic on the intersecting street. This constitutes an altogether unnecessary delay and slows up vehicular movement. The detrimental effect of synchronized signals was practically admitted by the Police Commissioner when he asked for an appropriation to extend the system. He said that the installation of lights on any street caused the drivers of vehicles to avoid that street and use a parallel thoroughfare. This he wished to discourage by equipping the parallel street with similar signals. Such lack of adjustment of the system to meet actual needs causes disgust with the whole arrangement and frequent disregard of it.

This leads directly to the third weakness of the system now in vogue in New York City—its slipshod enforcement by the police. Where traffic officers are present, they sometimes enforce immediate compliance with the signals, sometimes allow vehicles to "steal" a little on the light, and not infrequently make no attempt at all to see that the signals are obeyed. Where no traffic officers are present, a majority of the drivers of vehicles obey the lights, but a large minority ignore them. From an accident standpoint this creates a hazard at least as great as, if not greater than, that which would exist without the signals.

* Associate Editor, *Electric Railway Journal*, New York, N. Y.

Thus, it appears that two things are urgently needed to improve traffic control in New York City: First, an engineering study to determine the most efficient type of signal system and how it should be operated; and, second, more careful enforcement of traffic regulations.

T. KENNARD THOMSON,* M. AM. SOC. C. E.—For more than sixteen years the speaker has done some work nearly every day on a plan which, it is hoped, will not merely reduce, but almost eliminate, the automobile hazard.

It is almost needless to point out the intolerable condition of New York streets—where one can often walk a mile quicker than he can ride—to say nothing of the accidents. There is no reason why the city should not have buildings as high as it wants, as many people as can occupy the buildings without congestion of the sidewalks, streets, or transit lines, and without the enormous cost of subways, which now is about $9 000 000 per mile.

The city cannot raise money to build enough subways to eliminate the congestion. The remedy which the speaker has been advocating for many years is multiple streets (one over the other) with separation of different kinds of traffic and the ultimate elimination of subways for passengers. Subways never should have been built, for five streets above ground can be built for the cost of one subway.

Fig. 5 shows a plan for three elevated streets (not viaducts) above the present grade. The ground or present street level of the avenues is to be reserved for trucking; the next level for transit lines, with the local cars running close to the sidewalks, allowing the passengers to observe well-lighted shop windows all the way. This level, as well as the one above it, will have elevated sidewalks so that one cannot step in front of automobiles or transit lines.

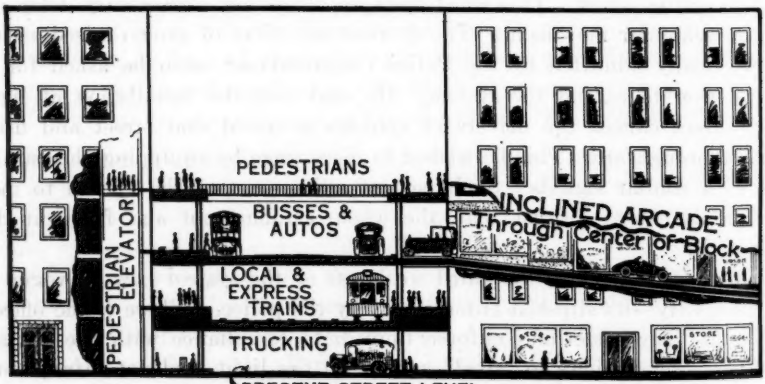


FIG. 5.—PROPOSED RAPID TRANSIT PLAN.

Access from the third or automobile level to the present ground line will be by inclined arcades through the centers of the blocks, giving four sidewalks where two now exist. At first, these inclined arcades should be placed

* Cons. Engr., New York, N. Y.

a mile or two apart. Later, they may be installed wherever the property owners are willing to beg or pay for such accommodation, which means, eventually, every block.

The most valuable floor of a building is the ground floor— this plan gives four ground floors. By leaving the present cross streets as they are, there would be natural ventilation every 200 ft. on the two lower levels of the improved avenues. These levels would have perfect artificial lighting. The third, or automobile, level, and the fourth (sidewalks only), would have sunlight.

When first the speaker started these plans most people laughed at him. Later, there were various attempts to appropriate his plans bodily.

The speaker is always interested, although sometimes skeptical, in discussions on zoning. To advocate, as some very prominent men have done, that areas within New York City be laid out with very narrow streets, which will never permit thoroughfare traffic, would result in the value of the land being restricted to that for a two or three-story building—would be confiscation.

Even the multi-millionaires scrap their marble mansions when the value of the land becomes high enough. One man built what was then the highest building in New York, and at once tried to convince people that no one else should be allowed to put up a high building. When zoning becomes confiscation it is time to pause.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

WATER SUPPLY FOR ARMY RAILWAYS IN FRANCE

Discussion*

By William G. Atwood, M. Am. Soc. C. E.

William G. Atwood,† M. Am. Soc. C. E. (by letter).‡—This paper is a valuable addition to the record of the engineering activities of the American Expeditionary Forces and illustrates, as all thoughtful papers must, the woeful waste caused by unpreparedness. The lack of any plans for army organization, which included the engineering and transportation phases of the operations of an overseas army, made necessary the continual re-organization mentioned, until a satisfactory one was developed. The delays and losses of time resulting can be imagined, but must have been experienced to be fully appreciated. They were unavoidable under the conditions. Papers like this will put on record the experiences and needs of the multitudinous services of a large army in such a way that similar experimenting will not have to be done in the face of an active enemy in the next emergency.

If it had not been for the inspired foresight of Col. Wilgus§ when he prepared the famous Requisition No. 6 only a short time after the 1st Division landed in France, conditions would have been much worse than they were. That a requisition could be prepared providing for automatic shipment of the enormous number of items required for the construction, operation, and maintenance of the army transportation net based on the number of men in the field was a wonderful piece of work. To have that requisition so complete that it was successfully made the basis for the supply of the transportation needs of the Army during the entire period of hostilities was still more wonderful.

---

* This discussion (of the paper by Paul M. LaBach, M. Am. Soc. C. E., published in April, 1927, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, April 7, 1927.

§ *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 547.

It was unfortunate that the Water Supply Service of the Transportation Corps could not have been organized sooner, since it would have been very helpful had the Section Engineers known what would be needed in the way of roadside water stations; but this condition was no different from those pertaining to the facilities of any other branch of the Army.

The general water supply problem was one of the most serious among the many that had to be solved. There was always apparently water enough to make mud, but little for any other purpose. When the original plans for the development of the St. Nazaire-Montoir Area were submitted by the 17th Engineers (Ry.) in October, 1917, the doubtful factor was the water supply.

The Montoir Yard, according to Table 1,* would require a new supply of 427 000 gal. per day, but the city, camps, hospitals, and ships were estimated to require 2 500 000 gal. more, with an existing plant in the city furnishing a maximum of 375 000 gal. It was necessary to construct a number of reservoirs, some of them 25 miles away, to secure sufficient storage; and when the water had been impounded the problem of getting it to the point of use was still more serious in view of the lack of pipe and pumps. About 5 miles of 24-in., and 2 or 3 miles of 12-in., pipe were all that could be secured. The final pumping capacity, including boosters, was about 10 000 000 gal. per day. The railroad consumption at the Port of St. Nazaire and Montoir was roughly one-fifth of the total, and while the railroad use was of equal importance with the others it would have been "putting the cart before the horse" to have placed the design and construction of this plant in the hands of the Railroad Water Service, had it been organized at that time.

In the Montoir Area, roughly 2 by 6 miles, where all the construction was in virgin territory, the six railroad water tanks and engine house were supplied from the same mains that served the camps and the new wharf. In the other terminals in Base Section No. 1, Nantes-St. Luce, La Rochelle, La Pallice (later Base Section No. 5) and Saumur, the latter was the only point at which the railroad and other activities were so separated as to make the railroad water supply an individual problem and in that case the solution was simple.

All the roadside water stations were relatively small, but some of them were hard problems because of the difficulty of developing adequate supplies with available equipment and material. Aside from the tanks and water columns they required no material or equipment not common to other water supply installations, and a number of railroad water tanks were used for other purposes.

The theory of army organization does not differ from that of any other well-organized activity. There is complete centralization of authority as to policy and complete decentralization as to execution, but always there is a clear line of authority and responsibility.

The author would change this procedure so far as the water supply organization is concerned, and in this the writer believes he is wrong. The purpose of all activities of an army is to defeat the enemy and the duty of each part of

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 553.

the organization is to carry out the plans of headquarters to this end; for that reason the line of authority and consequent responsibility must be clear. One of the greatest steps for the improvement of conditions relating to construction in the A. E. F. was made when the General Purchasing Board under General Dawes placed the different items of supply in the hands of some one branch of the Army and made it responsible.

The writer believes that the final organization of the construction and operating activities of the A. E. F. was good. The Transportation Corps, Medical Corps, Quartermaster Corps, Air Corps, and others presented their approved plans to the Director of Construction and Forestry, and they were passed along for execution to the Section Engineers, each in his respective Section. The Section Engineer controlled all available labor and supplies in his Section and allocated them in conformity with the priority schedules furnished him. Any other plan is bound to be wasteful of both material and labor, neither of which were or ever will be adequate.

The Engineer of Water Supply, Transportation Corps, should be a specialist and should have adequate assistance, but he, as well as most other specialists, should be Staff Officers. The Section Engineers should have, and did have as far as Base Section No. 1 was concerned, such corresponding staff specialists as were needed. The section organization knows local conditions and, with proper information as to requirements and general design, should be able to construct adequate facilities much more efficiently and with much less waste of labor and material than a number of small independent organizations.

No units were organized for base section work in the A. E. F., but the two most important Base Sections, Nos. 1 and 2, were continually supervised from August, 1917, to the Armistice by the 17th and 18th Engineers (Railway), respectively. The writer believes as a result of his experience as Section Engineer of Base Section No. 1 that organizations of this type are fully competent to supervise installations for all branches of the service once they are informed of the requirements, and that independent individual organizations of specialists are out of place, except as designers or, in Army parlance, staff departments. The use of separate construction organizations and sources of supply for various branches of the Service was tried in the A. E. F. and found wanting.

The writer, therefore, finds himself in disagreement with the author on most of his conclusions. He believes that the Engineer of Water Supply, Transportation Corps, should furnish a statement of the requirements in as much detail as possible. This will reach the Section Engineer with an indication of the desired priority and from that time on the Section Engineer is responsible for furnishing the necessary facilities at the proper time. He knows what labor and material he has or can make available, and it is his business to follow as closely as possible the plans furnished him. At the proper time he must furnish an operating unit that will perform the required service whether or not it conforms to the plans in all respects.

The writer agrees with the author's Conclusion 5,* regarding the maintenance and operation of the water stations once they are completed.

The delays on account of the necessity of sending formal communications through channels are very real, but many of them can be avoided by informal conference and agreement. In the A. E. F., work was delayed on account of the necessity for securing approval from the French; but this was not an unreasonable requirement on their part, although it was very trying at times. The writer remembers one case when approval of the plans for an arch dam for a very badly needed hospital water supply was long delayed because of doubt in the minds of the French engineers as to the adequacy of the structure. When they finally authorized the work to go ahead it was with the expressed hope that the Americans would never find it necessary to fill the reservoir. At that very moment there was 10 ft. of water behind the dam and it was rapidly filling.

The matter of army engineering organization is one which all civilian engineers should study because it will be the case in the next war as in the last one that the Reserve Officers will have to furnish the largest proportion of the personnel, and they should have as complete a picture as possible of the requirements and difficulties. Railway water supply is an extremely important item and this paper very fully and correctly shows its difficulties.

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 561.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

PROBABILITY OF FLOOD FLOWS

Discussion*

By Messrs. H. Alden Foster, W. P. Creager, A. Streiff, and Sigurd Eliassen

H. Alden Foster,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author has presented several interesting examples of the application of theoretical probability curves to the analysis of flood flows. The results in general are quite satisfactory, except in the case of Fig. 2§ which shows flood peaks on the Coosa River. For this river, the author states that he was unable to fit the theoretical curve to the plotted data. The writer has attempted to check the plotting of the theoretical curve, using the values for $c\ v$ and $c\ s$ given by the author. In making this check, the mean flood value was computed as closely as possible to correspond with the data plotted in the diagram. The resulting theoretical curve agrees very well with the integrated frequency curve shown by the author. It appears, therefore, that the discrepancy in this case must have been of an arithmetical nature, rather than in the method itself.

The author, in plotting his data, locates the largest flood at $\frac{100}{n}$ % of the time, n being the number of items considered. Thus, with 294 floods, the first point is plotted at 0.34% of the time. With this method of plotting, there will be no point plotted at 0%, but the last point will fall at 100 per cent.

This method of plotting is suitable, provided only the actual data are to be considered. When, however, the plotted points are used to construct a smooth curve, which is then assumed to indicate the general law governing the data, the method of plotting must be revised. The use of a smooth curve involves

* This discussion (of the paper by F. G. Switzer, Assoc. M. Am. Soc. C. E., published in April, 1927, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† With Parsons, Klapp, Brinckerhoff & Douglas, New York, N. Y.

‡ Received by the Secretary, May 5, 1927.

§ *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 566.

the assumption that there are an infinite number of points available, the curve itself representing a mathematically continuous function. The original points, then, must be considered as only a very small part of the total number of possible items that could be plotted, if the data were sufficiently extensive. Each of the original points, therefore, should be treated as a typical value, or an average of many possible values; and each original point should be plotted at the center of its respective section of the time scale, instead of at the end of this section.

The following rule, therefore, should be followed in plotting such data: If n is the number of items in the record, and m is the sequence number of any particular item (when arranged in order of magnitude), the percentage of time at which this item is plotted will be $\frac{100\,(2\,m - 1)}{2\,n}$.

W. P. CREAGER,* M. AM. SOC. C. E. (by letter).†—Studies of the frequency of floods have been made by two general methods: (1) By the consideration of annual peaks; and (2) by the consideration of the actual number of "floods" during the period of record irrespective of the date on which they occurred.

In the second method, a "flood" is defined as any increase in flow above an assumed fixed basic stage. The choice of the basic stage influences the number of floods to be considered, since obviously an extremely high or an extremely low basic stage would result in the consideration of very few floods. Theoretically, the best basic stage is that which results in the greatest number of floods, since this gives the greatest number of data and hence the most accurate result. Unfortunately, however, in many rivers, the basic stage resulting in the greatest number of floods corresponds to a flow so low as to be entirely below the flood class, and some other method of fixing the proper basic stage must be devised.

Considerable may be said regarding these two methods. It is contended that the use of the annual peak method results in the exclusion of a number of floods which should be considered in a true probability study. On the other hand, some claim that the yearly peak method involves the use of a definite record of events capable of exact analysis, while the basic stage method is a limited series, that is, limited by the basic stage to the exclusion of floods or flows below that stage which are in reality in excess of normal.

It seems to the writer that, if floods are truly seasonal, the occurrence of two large floods in one year would be a single phenomenon. If, however, floods are not seasonal and are as apt to occur in one part of the year as another, there would have been two events in that year, both of which should be included in the series.

While many streams tend toward seasonal floods, there are also many in which floods are distinctly not seasonal, although they may be more apt to occur in the spring when the snow is leaving the water-shed. The greatest recorded floods on the Black River, at Watertown, N. Y., occurred in December. The greatest recorded flood on the Deerfield River, at Charlemont, Mass.,

* Vice-Pres. and Chf. Engr., The Northern New York Utilities, Inc., and Power Corporation of New York, Watertown, N. Y.

† Received by the Secretary, May 13, 1927.

occurred in July. The use of daily records certainly gives a definite series and the author has shown that, for the Coosa River, the determination of flood frequency by daily flows agrees almost exactly with the basic stage method.

At any rate the use of the basic stage method results in a greater frequency of flood flows than the yearly flood method and, therefore, is more conservative. The calculated 1 000-year flood on the Tennessee River, at Chattanooga, Tenn., is 455 000 sec-ft. by the yearly flood method and 475 000 sec-ft. by the basic stage method, using a basic stage of 100 000 sec-ft., or 179 floods in a period of 41 years.

By the study of the flood probabilities of three rivers, the author has determined that, for these rivers, floods of a given frequency vary as the 0.6 power of the drainage area. He states that with this limited data the equation can have only local application. The writer believes that, in general, floods will vary more nearly with the 0.5 power of the drainage area, although the relation may vary considerably for streams of different characteristics.

A. STREIFF,* M. AM. SOC. C. E. (by letter).†—Estimating probable maximum floods is a fundamental, and yet an unsolved, problem in the design of hydraulic structures. About fourteen years ago the statistical methods of using the frequency curve for this purpose was proposed by Allen Hazen, M. Am. Soc. C. E., and used‡ by W. E. Fuller, M. Am. Soc. C. E. The flood flows from one record, or a combination of records, are tabulated and a duration curve is made. This duration curve is rectified by using a special variable ordinate scale, such as Mr. Hazen's "probability paper" and, then, in the words of Mr. Hazen,§ "the reasonable projection of the mean line may be taken as giving an idea of the probable frequency of floods greater than those covered by the record."

The fundamental question is whether Mr. Hazen's assumption is correct. Does the extension of the duration curve have the meaning attributed to it? This question is not limited to flood flows, but applies equally to rainfall, run-off, and storage, to all of which the duration curve has thus been applied.

Other engineers give little credence to this method. Robert E. Horton, M. Am. Soc. C. E., states‖ of the frequency curve: "Such formulas seem to the writer defective"; and the comment¶ of D. W. Mead, M. Am. Soc. C. E., is: "Such calculations must not be taken too seriously". The statistician, Professor G. Udny Yule, believes** that on account of the presence of cycles frequency computations based on 20 or 30-year records cannot be certain. Discussing the record of 125 major floods on the Danube River since 1012, the late H. M. Chittenden, M. Am. Soc. C. E., stated:†† "A noteworthy feature of the record is the occurrence of flood years in groups. In nearly the entire period, high flood years were bunched together, showing that precipitation

* Cons. Engr., Bangor Hydro-Elec. Co., Jackson, Mich.
† Received by the Secretary, June 1, 1927.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.
§ *Loc. cit.*, p. 632.
‖ *Journal*, New England Water-Works Assoc., Vol. XXXVII (No. 1), p. 23.
¶ "Hydrology," p. 570.
** "Introduction to the Theory of Statistics."
†† *Engineering News*, Vol. 60 (1908), p. 467.

moves in cycles." In that case, therefore, the frequency curve must vary according to the place in the scale of time where the record is taken.

The fundamental condition on which the derivation of the normal law of errors by Gauss and Bessel is based, does not prevail in the case of flow records. Probability paper is used with full knowledge of this point. That scale is merely used to rectify the curve as much as possible, and this can also be done by other scales. The underlying idea is that the curve is rectified to such an extent that it (presumably) can be prolonged without appreciable error. This, of course, is not exactly a solution, but merely transfers the difficulty to the prolongation of the scale, the accuracy of which is unknown.

Another and fundamental difficulty is called, in statistics, the theory of sampling. Can the records be regarded as constituting correct samples? Mr. W. E. Fuller states:*

"As most of the data were obtained during the last 10 or 20 years, objections to this method of analysis may be raised on the score that weather conditions may have been different during other periods in the past, or may be different in the future. Certainly, if weather conditions throughout the country were subject to permanent changes, this objection would be well taken. There is no evidence, however, that this is so."

And, again: "* * * it seems that periods of 10 years or more give a fair average of the conditions which have existed in the past and will probably exist in the future." On the other hand, Mr. Fuller also shows that such short records may be much in error.†

Nevertheless, adopting this theory, conclusions are drawn regarding the probability of occurrence covering an unlimited future. Thus, Mr. Hazen uses artificial 300-year records.‡ Mr. Fuller considers 1 000-year records; while Professor Switzer extends the considerations to 50 000 years.§ These extensions are based on records of 20 to 50 years in length.

Consider, however, that only 12 000 years ago the earth was in the Ice Age. Who shall conclude from a 50-year record of the present day anything regarding the floods of the Tennessee River during the Ice Age? The recession of Niagara Falls,‖ the uncovering of Toronto, Ont., Canada,¶ the recession of St. Anthony Falls,** all indicate a period of from 12 000 to 35 000 years since the recession of the last ice sheet. Since then many changes have occurred. Somewhat more than 14 000 years ago, as evidenced by the thickness of alluvial muds at Cairo, the climate of North Africa was moister and colder than now.†† The occurrence of *Rhododendron ponticum* in the calcareous

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 574.

† *Loc. cit.*, p. 684.

‡ *Loc. cit.*, p. 592.

§ *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 563.

‖ Geologic Atlas, Folio No. 190, U. S. Geological Survey, F. B. Taylor and E. M. Rindle, 1913.

¶ "An Estimate of Post-Glacial and Interglacial Time in North America," by A. P. Coleman, International Geology Congress, 1913, pp. 435–450.

** "Beginning and Recession of St. Anthony Falls," by T. W. Sardeson, *Bulletin*, Geological Soc. of America, Vol. 19, pp. 29–52.

†† "The Glacial Period and Climatic Changes in Northeast Africa," by W. F. Hume and J. I. Craig, Rept., British Assoc. for the Advancement of Science, 1911, pp. 382-383.

tufa in the Island of Skyros, in Greece, is conclusive evidence of a heavier rainfall than at present.*

After the retreat of the ice a great amelioration of the climate set in, as evidenced by the so-called Alleröd beds in Scandinavia.† The laminated clay layers accompanying the retreat of the glaciers contain remains of Arctic plants; they are covered by the Alleröd beds, which contain flora and fauna of a warmer climate. Thereafter, the climate again became colder, as newer deposits over these Alleröd beds, again containing Arctic plants, testify.

In North America, apparently, the same oscillations took place. Certain marine mollusks, whose present northern limit is Cape Cod, Massachusetts, once spread into Casco Bay, in Maine.‡ Sub-fossil marine shells at Boston, Mass., indicate a climate similar to that off the Virginia coast at present.§ A fish weir found in the deposits shows that man lived there during the warm period, possibly about 3 000 years ago. Composition and stratigraphy of peat bogs in Ohio seem to indicate alternate dry and wet periods since the recession of the ice.‖ The earlier distribution of mammals and plants make it probable that the southern shores of the Great Lakes enjoyed a climate similar to that of Tennessee and Arkansas at present.¶ It is possible that during this warm period, following the retreat of the ice, the Great Lakes were lower than at present. The Ausable and Manistee Rivers in Michigan have excavated since then a deep valley in the glacial moraine and overwash plateau that covers the northern part of the Southern Peninsula, 600 ft. deep; but deltas are missing where they enter the Great Lakes.

In the Southwest the moisture has undergone perceivable changes in post-glacial times, as recorded by many changes in lake levels and changes in civilization.** Owens Lake, California, overflowed about 2 500 years ago, as did Pyramid Lake, Nevada; while Walker Lake and perhaps, also, Winnemuca Lake have been entirely dessicated. Huntington concludes from the ancient ruins in Nevada that the climate there some thousands of years ago must have been considerably moister than at present. The famous terraces of the phantom Lakes Bonneville and Lahontan in the Great Basin are silent witnesses of the general dessication of that region since the Ice Age. The record of many oscillations is written in the succession of beaches. Such oscillations occur at present. In 1869 the Great Salt Lake was 420 sq. miles greater than in 1850, when it measured 1 750 sq. miles. The original Lake Bonneville had an area of 19 750 sq. miles. The maximum depth was 1 050 ft. where now it is 30 ft.††

In recent times the oscillations in climate have been many, although no permanent changes occurred of considerable magnitude. Huntington records

* "*Rhododendron Ponticum* Fossil in the Island of Skyros in Greece," XI Inter. Geological Cong., Stockholm, Sweden, 1913.

† "Post-Glacial Climatic Changes in Denmark," by Valdemar Nordmann, Inter. Geol. Cong., Stockholm, Sweden, 1913, pp. 311–328.

‡ "Changes in Climate in the Maritime Provinces After the Maximum of the Last Glaciation," by G. F. Matthew, Inter. Geol. Cong., Stockholm, Sweden, 1913, pp. 377–380.

§ "Post-Glacial History of Boston," by H. W. Shimer, *American Journal of Science*, Ser. 4, Vol. 40, pp. 637-642.

‖ "Peat Deposits and Their Evidence of Climatic Changes," by A. P. Dachnowski, *Botanical Gazette*, Vol. 72, 1921, pp. 57–89.

¶ "The Life of the Pleistocene or Glacial Period," by F. C. Baker, Univ. of Illinois, 1920.

** "Quaternary Climates," by Huntington and Antevs, 1925.

†† "Lakes of North America," by I. C. Russell.

a dry period about 650 A. D., as shown by preserved inscriptions on old structures at the Caspian Sea, recording an extreme low level. This dry period is also visible in the Sequoia record.* The vine once grown in Brittany and along the Bristol Channel, no longer ripens there. Iceland and Greenland have become colder since the Fourteenth Century, for large trees no longer grow on Iceland, and once inhabited valleys of the Greenland coast are now filled with ice.† In the Fourteenth Century the record of a period of climatic stress is amply written in history. The entire Baltic was frozen over, as well as the rivers of the German coast; and tremendous floods, such as the historic calamity of the Biesbosch, in Holland, when seventy-two villages were wiped out by the rising waters, are on record.

Professor Huntington concludes: "The greater the amount of detail which we are able to obtain, the fuller become the evidences of a great number of climatic pulsations".

In view of these variable conditions, it is impossible to extend frequency curves so as to give probability of occurrence measured in thousands of years. Neither short records nor very long epochs can be explored by means of a frequency curve. Is there any tangible evidence that intermediate values of, say, 25 to 100 years, would be reliable? Is it safe to assume that a duration curve of, say, 40 years would be approximately the same for the next 40 years, or could this be expected to be the case with two consecutive 100-year records? By investigating the cyclic variations it may be shown that such lengths of record, too, are unreliable, and consecutive records giving divergent duration curves can easily be found.

Take the figures of the Tennessee River, investigated by Professor Switzer. It so happens that in this record, extending from 1874 to date, the course of the Brückner cycle and the "Secular" cycle,‡ on which it is superposed, is plainly written. The first part of the record coincides with a wet phase of the Brückner and of the Secular cycle (Fig. 5), while the last half of the record is characterized by the low phase of the latter cycle, which to a considerable extent neutralizes the recurring wet phase of the Brückner cycle. Hence, dividing the record in two parts of 25 years, two different duration curves are obtained (Fig. 5). It is interesting that the duration curves of the maximum yearly floods follow the duration curves of the average yearly flow—an important relation, indicating the greater probability of large floods during generally wetter periods.

Perhaps better results would be obtained with still longer records. These are not to be found in North America, but in Europe. The cycles again show which records to select, in order to obtain divergent duration curves. It so happens that while the Brückner cycle changes phase with geographical location, there is ample evidence that the longer "Secular" cycle prevails in North America and Europe in much the same way. Hence, the condition in Europe during the last 40 years should have been generally dryer than previously. Dr. J. Epper, former Director of the Hydrometric Bureau in Switzer-

* "The Pulse of Asia," by Ellsworth Huntington.
† "Physical Geography," by Elisée Réclus.
‡ *Monthly Weather Review*, November, 1926.

land,* observes that in the first 70 years of the Nineteenth Century twenty major floods occurred in Switzerland. In the latter part of the Nineteenth Century a marked retrograde movement of the Swiss glaciers took place, also visible in the reduced summer flow, which is the high-water period of the

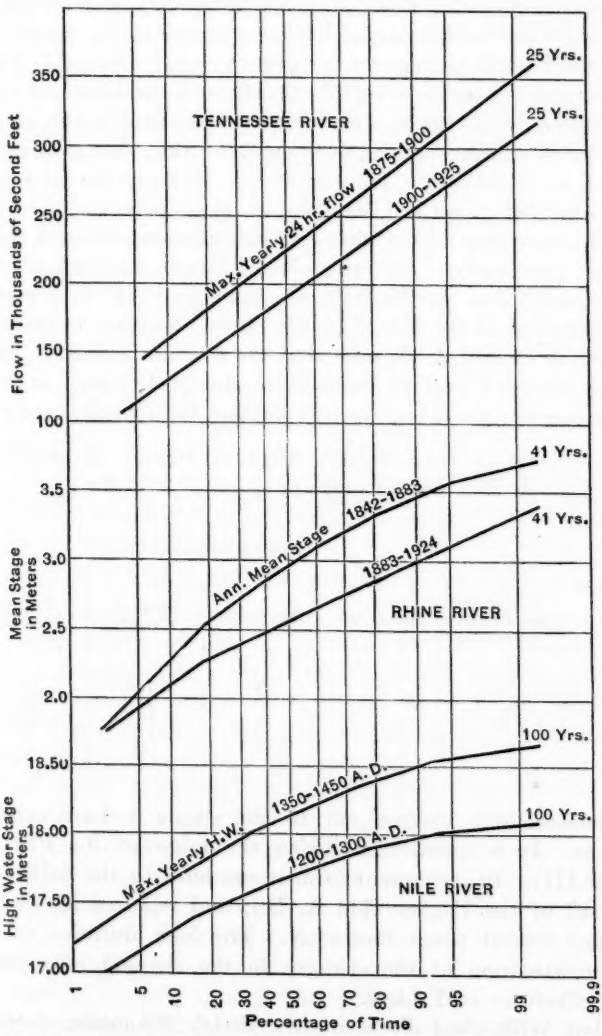


FIG. 5.—DURATION CURVES FOR VARIOUS EPOCHS ON TENNESSEE, RHINE, AND NILE RIVERS.

Alpine rivers. This condition is now nearing the end; the last report of Professor Mercanton on the observations of glaciers carried on by the Swiss Alpine Club,† gives for 1926 a net advance of the Swiss glaciers. Selecting two consecutive, 41-year records of the Rhine—from 1842 to 1883 and from 1883 to 1924—Fig. 5 gives different duration curves.

* "Die Entwickelung der Hydrometric in der Schweiz," Swiss Federal Hydrometric Bureau, 1907.

† "Die Alpen," Schweizerischen Alpen Club, May, 1927.

Then, perhaps, identical duration curves might be obtained by taking still longer records, for example, two consecutive 100-year records. Several such records are available, on the Rhine, the Seine, the Danube, the Po, and the Nile. The Nile record will be selected. The ancient Egyptians, to whose fields the annual floods of the Nile brought fertility and abundant harvests, early recognized the importance of having a record of the water stages in the river, and established permanent gauges in many places.* The difference between high and low water is usually 25.5 ft., and the high water is singularly constant, varying from year to year not more than from 3.5 ft. above, to 5.5 ft. below, the mean. However, small changes in this elevation meant famine, good harvest, or inundation. The population awaited with anxiety the rise to 16 cubits on the Nilometer (El Mekyas), at which point the diversion into the storage basins took place. A depth of 12 cubits meant reduced crops, while 18 cubits spelled inundation. No rain falls in Egypt, and agriculture is, therefore, wholly dependent on the Nile, the supply of which, derived from the tropical "Mountains of the Moon", seldom fails. Failures resulted in famines, as that occurring in 1181 A. D., related by the Arabian author, Ibn Iyas.† Such misery then prevailed that a wholesale exodus took place; cannibalism was practiced among the poor, and secretly became fashionable among the rich.

TABLE 6.—RANGE OF NILE FLOODS SINCE ANTIQUITY, CLASSIFIED IN TERMS OF HEIGHT IN CUBITS (1 CUBIT = 1.77 FT.).

Classification of floods.	During the Time of					
	Moeris, 900 years before Herodotus.	Herodotus.	Petronius.	Omar-el-Katteb.	Kalka Shandi.	Present.
Bad	.....	10	11	12	14	17
Passable	.....	11	15	14	15	18
Good	9	.....	16	16	15.5	22
Very good	.....	.....	.....	17	17	22.5
Inundation	.....	.....	.....	18	18	24

The Nilometer (El Mekyas), at Rodah, seems to have existed in very ancient times. It is mentioned during the reign of the Pharaoh Moeris (Amenemhat III). In its present form it was built by the Sultan Souleyman in the year 97 of the Hegira (715 A. D.), and repaired by the Caliph El-Maimoun and several times thereafter. The long duration of the record makes an investigation of the changes in the channel imperative. These are given by Chélu as in Table 6.

Sir William Willcocks,‡ discussing the Rodah Nilometer, states:

"When the gauge was constructed, a reading of 16 cubits (1 cubit, or Arabian yard, = 1.77 ft.) meant the lowest level at which flood irrigation could be insured everywhere. The level to-day is 20.5 cubits on the gauge and the difference between them is 1.22 meters. As 1026 years have elapsed since the reconstruction of the gauge it means a rise of 12 cm. per 100 years. This is slightly under the rise calculated at Assuan by the French savants."

* "Le Nil. le Soudan, l'Egypte," by A. Chélu, Paris, 1891.

† "Mémoire sur l'Historie du Nil," by S. A. le Prince Omar Toussoun, Cairo, 1925.

‡ "The Nile in 1904," by Sir William Willcocks, Lond., 1904.

Selecting a portion of the record between 1200 and 1450 A. D., it may be seen that the correction to be applied is 58 ft. There are three long time Nile records available, preserved in the writings of Arabian scholars: (1) Ibn Abik; (2) Aboul Mahasin; and (3) Shibab El-Din El-Higazi.* They are complete from 622 to 1470 A. D., but, unfortunately, the remaining centuries to date are partly missing. The modern records since 1860 have a limited value, for the gauge is influenced by the weirs of Damietta and Rosetta built by the French engineer, Mougel Bey, and by the storage now practised by the British engineers. The records of the scholars differ somewhat, but for the investigation of cycles the writer finds the differences unimportant.

The Rodah gauge was a stone pillar, housed under a mosque-like cupola, measuring 80 ft. from the base of the gauge to the top of the cupola. The data for the two duration curves of Fig. 5 were read from this gauge, during the years, 1200-1300 A. D. and 1350-1450 A. D. It may be seen that these two duration curves are wholly different, showing that even 100-year records may be insufficient

In these examples the high and low parts of the cycles were purposely selected, and hence such differences will not always be found. In an unknown record the middle portion of the cycle might be contained; but unless these cycles are investigated, uncertainty of the meaning of the duration curve will prevail, and their application should, in the words of Professor Mead, "not be taken too seriously."

SIGURD ELIASSEN,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The author has done the Society a service by calling attention to certain considerations with regard to the interpretation of flood flow probability curves. Engineers are often in a quandary as to what data to include when constructing such curves—whether or not to base them on the maximum yearly flood flows alone, for the reason that some of the secondary floods during a year may be larger than the maximum floods of other years. The author's solution of this problem is not new. However, a discussion of it seems profitable as there are several features involved, which are not mentioned in the paper.

As long as the yearly maximum flows only are used for the construction of the probability curve (duration curve) its interpretation is simple enough, namely, that a given flood is likely to be exceeded or not exceeded (according to the manner in which the curve is constructed), in a certain percentage of the time, which in this case may be interpreted as a certain number of years out of 100 years. The unit time period is here 1 year. If sufficient data are available the maximum flood occurring every second, third, fourth, or fifth years, etc., can be selected as the basis for the curve. The unit time period will then be the period selected. If the curve is to be interpreted on the 100-year basis, the percentage value read on the time co-ordinate will have to be divided by the number of years in the chosen time period. Thus far,

* "Mémoire sur l'Historie du Nil," by S. A. le Prince Omar Toussoun, Cairo, 1925.

† Engr. in Chg., Topographical and Hydrometric Survey Dept., Chihli River Comm., Tientsin, China.

‡ Received by the Secretary, June 6, 1927.

this is clear enough, because the unit time period was a selected one and not subject to undefined irregularities.

In the other direction, however—that is, selecting more floods than the maximum for each year—the interpretation of the curve becomes complicated. The case is not satisfactorily settled by using as the unit time period the number of years covered by the data divided by the selected number of floods. The selected floods have a certain distribution probability of their own, which may not necessarily follow the "normal law" and are scarcely ever a straight line function. To base the time unit on the mean interval between the floods, therefore, does not seem to be a correct procedure. Supplementary frequency curves are necessary in order to explain fully the first curve. Such curves should give the interval probability of floods, preferably of different magnitudes, if the data allow it, and other curves should give the probability of their duration.

There are other ways, however, of getting around the problem. One is to plot separate probability curves for the maximum floods in recurring periods of the year, such as the month, the week, or a certain number of days. In this way the time period is again chosen and can be directly transferred to fractions of years and hence be interpreted directly in terms of the 100-year period. An inspection of the data will show where it is necessary to make the smallest subdivision of time in order not to miss important secondary floods. A combination of such curves may be made.

If a probability (duration) curve is constructed for the whole range of flow, selecting as the time unit a period as small as practical (the second of time is evidently too small), then the curve will show the percentage of time the river is likely to exceed or fall below a given discharge. Such curves will give both the probable maximum discharge and the probable duration of its peak flow if the time unit has been made sufficiently small. For lower discharges the percentage of time as read from the curve will be made up of different periods and the interval between and the duration of these periods should be made subject to probability studies in the same manner as when floods other than the absolute yearly maximum are included.

The writer suggests that flood volumes also be treated in a manner similar to flood discharges. Such studies have been made with great profit in the case of deciding on the maximum flood volume to be controlled by detention basins for the Yung Ting River, in North China. Probability curves were constructed for flood volumes contained in 1-day, 2-day, 3-day, and 4-day floods.

When the records are short-termed, it is necessary to reflect somewhat on the manner in which the constructed curve is likely to be affected by additional data, high as well as low values. Where a new observation is inserted, the curve will be thrown slightly to the right on the right side and to the left on the left side of such point; but different shaped curves may be affected quite differently, either pointing to caution in the first conclusion drawn from the original curve or to confidence in its reliability. This is instructive practice and should not be omitted.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE EYE-BAR CABLE SUSPENSION BRIDGE AT FLORIANOPOLIS, BRAZIL

Discussion*

BY MESSRS. J. A. L. WADDELL, SPENCER MILLER, LLOYD G. FROST, C. G. EMIL LARSSON, AND CLYDE T. MORRIS

J. A. L. WADDELL,† M. AM. SOC. C. E. (by letter).‡—This bridge marks a forward step in the development of economics in suspension-bridge designing. All those concerned in its evolution are entitled to the hearty thanks of the Engineering Profession for their valuable contribution to the science and art of bridge building.

The structure contains a number of important innovations in bridgework, the principal ones of which are the utilization of the cables for forming a portion of the top chords of the stiffening trusses, the employment of rocker towers, the use of eye-bar cables, and the heat-treatment of carbon-steel eye-bars. It is true that these features of design and construction are not absolutely new; but they are unusual—especially in the United States—hence the designers and builders deserve much credit for their courage and enterprise in departing from the beaten paths.

The authors have pointed out many economies in making the cables serve as a portion of the top chords of the stiffening trusses; and the summation of these constitutes a saving in cost that must be of considerable magnitude. The authors state that "This utilization of the cable as the upper chord of the stiffening truss should preferably be limited to the central half of the span." From Fig. 2§ it appears that this limit has not been utilized, because there are eleven double panels in the central portion and eight in each side

* This discussion of the paper by D. B. Steinman and William G. Grove, Members, Am. Soc. C. E., presented at the meeting of the Structural Division, New York, N. Y., January 21, 1926, and published in May, 1927, *Proceedings*, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. (Waddell & Hardesty), New York, N. Y.

‡ Received by the Secretary, May 18, 1927.

§ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 710.

portion. By adding two double panels to the center and taking one from each side, the central portion would be made almost the half-span length; and, in the writer's opinion, the appearance would be improved, especially as the side portions could be made to appear almost identical with the halves of the central portion of the stiffening trusses. In Fig. 38 this suggested layout is shown beneath the layout adopted.

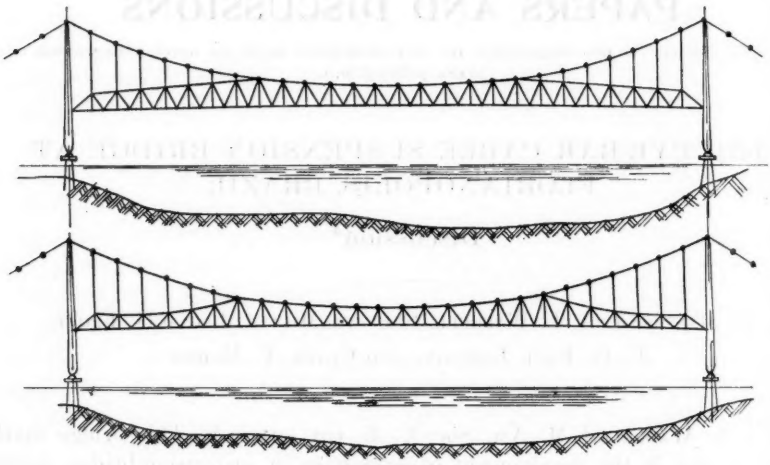


FIG. 38.

Possibly the designer of the structure had some good and sufficient reason for using the short central portion; and it would be interesting to learn what it was. It is evident* that he recognizes the improvement in appearance that could be effected by curving the outer portions of the top chords of the trusses. The writer, however, is of the opinion that if that had been done, the variation from parallelism of chords would not have been sufficiently pronounced in the adopted layout.

The writer is in agreement with the authors' claims of "esthetic values" for their structure, Fig. 2 (*a*), as compared with the previous design, Fig. 2 (*b*). The latter would have had a better appearance had one-half the suspenders been omitted, and had the truss been made a little deeper.

The esthetics of the cantilever design, Fig. 2 (*c*), could have been improved somewhat by accentuating the apices and the points of suspension, for the truss depths are too nearly uniform from anchorage to anchorage.

Rocker towers, compared with those fixed at the bottom, are both more scientific and more economical of metal; but it must be remembered that they will have to be guyed, or otherwise held in correct position, during erection. Whatever method may be adopted, some extra expense will be involved. The writer is of the opinion, however, that ultimately the hinging of the tower feet would prove to be economical.

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 721.

In May, 1920, the writer stated* that, under the then ruling prices, heat-treated, carbon-steel eye-bars having an elastic limit of 50 000 lb. per sq. in., could not compete with wire cables, even for spans as short as 1 000 ft., although the difference in costs was not great. Since then, however, the elastic limit of such eye-bars has been raised to 75 000 lb. per sq. in. This would reverse the conclusion for comparatively short spans; but, on the other hand, the cost of wire has been reduced about 25%, consequently it is still a mooted point as to whether wire cables or heat-treated, carbon-steel, eye-bar cables are the cheaper. A good test of this question for long spans will soon appear in the competitive bidding on the superstructure of the proposed Hudson River Bridge to be built for the Port of New York Authority. The competition in the case of the Florianopolis Bridge does not afford any conclusive comparison, because the manufacturers of the new type of eye-bars were greatly desirous of introducing them on the market.

The manufacture of eye-bars, so common in the United States three or four decades ago, is now a monopoly, hence low prices for heat-treated bars need not be anticipated, unless some rival company should start making them. In the old days, eye-bars were generally considered to be the cheapest part of the metal work of any pin-connected bridge, but nowadays the reverse seems to be the case.

As for the heat treatment of eye-bars, that question appears to have been settled by the American Bridge Company to its own satisfaction; but facilities for the inspection of the manufacture and the testing of full-length bars will have to be furnished before consulting bridge engineers in general will be willing to utilize the new product. It seems to the writer that a working intensity of 50 000 lb. is rather high for an elastic limit of 75 000 lb., and that, for a time at least, it would be more judicious and conservative to limit the stress to 45 000 lb. per sq. in.

The claims for greater rigidity in the new type of stiffening truss appear to the writer to be well founded, but he would like to ask whether any deflection tests have been made on the finished structure, and, if so, with what results. Incidentally, the "novel" pin-hole detail "patented by the American Bridge Company," was evolved by the writer† about fifteen years ago.

SPENCER MILLER,‡ M. AM. SOC. C. E.—The authors have made a notable contribution to engineering art in the design of the cableways for the erection of this bridge. It was a brilliant thought to use a large number of small cables grouped together, to act as a trackway, which afterward were available as hoist ropes. It would be interesting to know how many cables were really in contact with the main sheave pulleys in the traveling carriage itself, and how an equality of stress was provided on the part of the individual cables? Perhaps a cross-section of the cables as they actually appeared in practice, showing the dimensions and the character of the pulleys and their grooving, would be instructive.

* *Proceedings*, Engrs.' Soc. of Western Pennsylvania, Vol. 36, 1920–21, p. 418.
† "Bridge Engineering," pp. 611–612.
‡ Cons. Engr., South Orange, N. J.

Another question concerns the method of applying power for lifting loads from the barge. A diagrammatic view illustrating how the ropes were led from hoisting drums to the traveling carriage and fall-block, together with all leading sheave pulleys, would further add to the value of the paper.

LLOYD G. FROST,* ASSOC. M. AM. SOC. C. E. (by letter).†—In departing from conventional practice in suspension bridge design and construction, the authors have met the conditions peculiar to this problem in a signally efficacious manner. Further, they have rendered a distinct service to the Engineering Profession in establishing the practicability and economy of certain innovations in design and erection.

Most engineers specializing in bridge work find little occasion to investigate or design a long-span suspension bridge. In the light of conventional practice, with the massive towers, foundations, and expensive erection methods required, the use of the suspension bridge has been necessarily restricted to certain exceptional cases.

In the design and execution of the Florianopolis Bridge the authors have laid a basis for revising accepted practice in this regard. The exceptional economy of foundations, secured by the use of rocker towers, together with the generally improved conditions of rigidity and economy, point to a possibility of widening the range of use of suspension bridges to include many cases to which the conventional type of structure could not be adapted. Particularly does this seem possible in the case of highway structures with light loads and comparatively short spans, where the difficult and expensive foundations required in other types of bridges might be largely or wholly obviated by an adaptation of this variety of suspension bridge.

A study of this paper in connection with a projected structure has led the writer to make an estimate of the cost of a similar design in comparison with arch and cantilever types previously considered. Until the results of this and similar studies have been secured and thoroughly digested, no definite idea can be had of the effect of the Florianopolis project on suspension bridge design; but it is not improbable that the noteworthy economies attained will largely affect the design of suspension bridges in the future.

C. G. EMIL LARSSON,‡ M. AM. SOC. C. E. (by letter).§—As the design for the Florianopolis Bridge was made possible by the use of high-tension, heat-treated eye-bars, it may be of interest to note the development of the eye-bar industry.

About 1880 the first non-welded steel eye-bars were made. These bars had a minimum elastic limit of 28 000 lb. per sq. in., a minimum ultimate strength of 55 000 lb. per sq. in., and a minimum elongation of 15% in 10 ft. They fulfilled the construction requirements until in 1904 New York City undertook to build the Queensborough Bridge. As this long-span structure carried a heavy load, it was felt that a material reduction in cost could be accomplished

* Cons. Bridge Engr., with James W. Billingsley, New Orleans, La.
† Received by the Secretary, May 23, 1927.
‡ Asst. Chf. Engr., Am. Bridge Co., New York, N. Y.
§ Received by the Secretary, June 8, 1927.

if eye-bars could be made to have higher physical qualities than standard bars. After considerable discussion nickel steel was adopted as the material. Full-sized tests showed a minimum elastic limit of 47 000 lb. per sq. in., a minimum ultimate strength of 80 000 lb. per sq. in. and a minimum elongation of 10% in 18 ft. This type of bar was found to be quite expensive, and considerable study was devoted to the possibility of making a cheaper bar having about the same physical properties.

By 1914 study had led to the consideration of heat-treated carbon steel. Experiments were undertaken and after various difficulties were solved it was considered safe to state that heat-treated eye-bars could be produced of such quality that tests on full-sized bars would show a minimum elastic limit of 50 000 lb. per sq. in., a minimum ultimate strength of 80 000 lb. per sq. in., and a minimum elongation of 8% in 18 ft. Since 1914 full-sized bars to the number of 134 have been tested under the supervision of the purchaser's engineers. These tests indicate an average elastic limit of 57 800 lb. per sq. in., an ultimate strength of 86 200 lb. per sq. in., and a minimum elongation of 10.7% in 18 ft., showing that the specification proposed by the manufacturer was rather conservative. Of the 134 bars tested only one proved to be unsatisfactory to the purchaser, due to too small elongation, although even this item was considerably larger than that obtained on riveted tension members.

On account of the good results from this type of bar, it was considered practicable to produce eye-bars having still higher physical qualities. Therefore, a second series of experiments on full-sized bars was undertaken in 1921, with the result that eye-bars can now be made which are guaranteed to show a minimum elastic limit of 75 000 lb. per sq. in., a minimum ultimate strength of 105 000 lb. per sq. in., and a minimum elongation of 5% in 18 ft.* Almost always full-sized eye-bar tests have been made during the construction of a structure of any magnitude, while it has been quite unusual to make full-sized tests of riveted work. The knowledge of the strength of eye-bars is, therefore, much greater than that of riveted work.

Concern is expressed in the paper regarding the policy of secrecy covering the manufacture of these bars. One reason for this policy was that the manufacturer, being responsible for the quality of the bars, wanted to be free to adopt any variation in method that would improve them. The art of making heat-treated eye-bars has now advanced so far, and the process is so well established, that the policy of secrecy has been discontinued.

Clyde T. Morris,† M. Am. Soc. C. E. (by letter).‡—A number of important long-span bridges have recently been constructed or proposed. Three unusual features of the Florianopolis Bridge make this design a notable one: (*a*) eye-bar cables; (*b*) hinged towers; and (*c*) the combination of the stiffening truss with the central portion of the cables. The marked economies possible with the new type of stiffening truss and its increased efficiency seem evident; for these the authors deserve great credit.

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, Table 2, p. 723.
† Prof. of Structural Eng., Ohio State Univ., Columbus, Ohio.
‡ Received by the Secretary, June 20, 1927.

They state* that "the Florianopolis Bridge is the first suspension bridge in the Americas to be built with rocker towers." In 1914-15, a suspension bridge 700 ft. long was built over the Muskingum River at Dresden, Ohio, in which rocker towers were used (Fig. 39). The design was made by the writer, and the bridge was built under the supervision of Ralph H. Strait, County Engineer of Muskingum County, Ohio. This bridge also had eye-bar cables and an unusual stiffening truss. A brief description of it may be of interest here.

The first bridge over the Muskingum River at Dresden, was begun in 1851, and completed and opened to traffic in 1853. It was a private enterprise, constructed under a charter from the State of Ohio granted in 1848, and operated as a toll bridge until July, 1868, when it was purchased by Muskingum County and became a free bridge.

This old bridge was a suspension structure supported on four stone towers, with a central span of about 450 ft. Each of the four cables carrying the roadway was composed of 200 wires, No. 10 gauge, and was about 3 in. in diameter. It had wooden stiffening trusses and a wooden floor with a 17-ft. roadway, and was designed and constructed under the supervision of Mr. George Copeland.

The flood of March, 1913, rose above the tops of the stiffening trusses, and the current pulled the cables off the towers, thus wrecking the bridge. Local residents were strongly against changing the type of bridge. For years it had been the only suspension bridge in the State, and as the U. S. War Department objected to locating piers in the river, it was decided to use the suspension type for the new bridge.

The abrupt bank on the east side of the river in contrast with the wide flood plain on the west side, made an unsymmetrical bridge desirable. The river span was made 450 ft., with an approach span of 200 ft., suspended from the cable on the west side and a short 50-ft. simple span on the east. The east cables run directly from the tower to the anchorage.

Considerable study was given to the relative merits and costs of eye-bar and wire cables, and bids were asked for the two kinds, but both the preliminary estimates and the bids received showed the eye-bar cables to be the more economical. The material was open-hearth structural steel of 60 000 to 70 000 lb. ultimate tensile strength.

The erection was carried on during the winter when there was no river traffic (there is very little river traffic at any time of the year). This permitted falsework to be used for the entire length of the bridge. Probably the possibility of erection from falsework is partly responsible for the low cost of the eye-bar cable bridge.

Three-hinged stiffening trusses with parallel chords were used. According to the usual assumptions of constant shape of cable curve and equal hanger loads, such trusses are statically determinate, and their deflections are only slightly greater than those of a two-hinged truss. One objection to the three-hinged truss is the localization of the bending at the center hinge,

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 707.

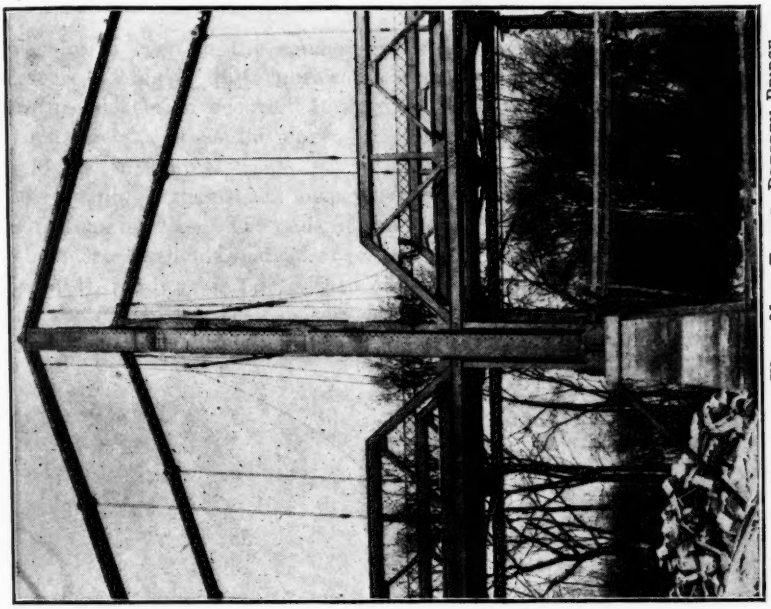


FIG. 40.—DETAIL OF WEST MAIN TOWER, DRESDEN BRIDGE.

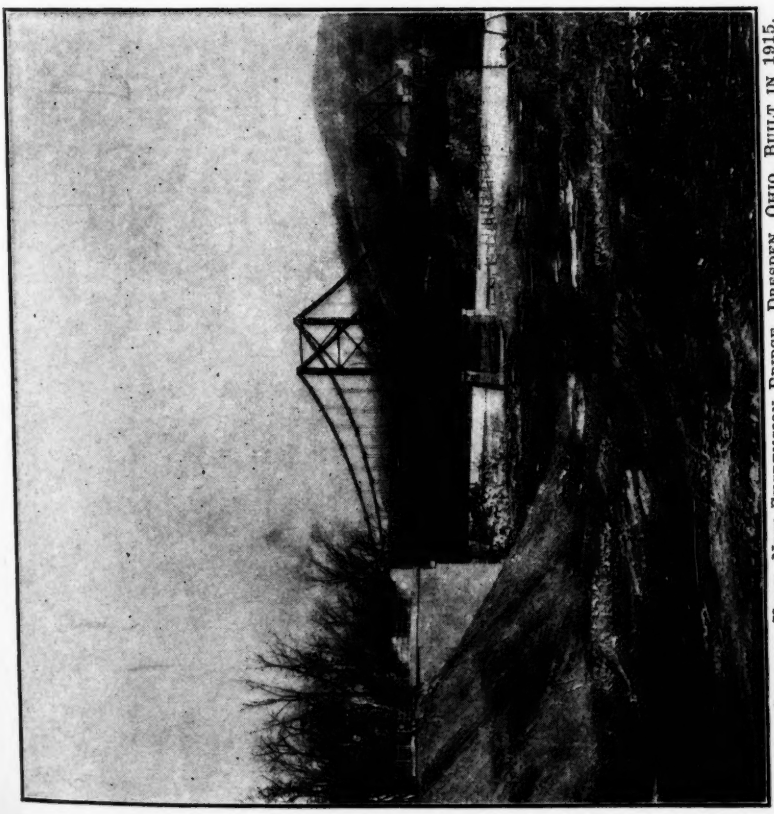


FIG. 39.—GENERAL VIEW, NEW SUSPENSION BRIDGE, DRESDEN, OHIO, BUILT IN 1915.





thus causing a cusp in the camber curve at this point under extremes of temperature. This is quite noticeable at mid-summer when the maximum temperature deflection occurs. It may be rendered unnoticeable by adjusting the hangers so that the curve is smooth at mid-summer, allowing the center to rise in winter.

The stiffening trusses are suspended from the cables by square hanger rods with turn-buckles. The ends of the trusses are supported at the piers on independent rocker bents instead of on the main towers. (Fig. 40.)

The 20-ft. roadway is paved with creosoted wood block on 3-in. creosoted plank. The bridge has been in service now for twelve years, and has proven entirely satisfactory. It is very rigid under traffic.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

CIVIL ENGINEERING FEATURES OF THE HELL GATE STATION

Discussion*

By Messrs. George A. Orrok and A. Streiff

George A. Orrok,† M. Am. Soc. C. E.—The Waterside Station of The New York Edison Company was designed and built in 1899. Although the machinery in this station has been enlarged three and one-half times since then, most of the original steel work is in place. An extra expenditure at the time of building (possibly $150 000) has saved several millions of dollars in these reconstructions.

The quantity of condensing water involves two questions—the summer water temperature and the water-rate curve of the turbine with respect to vacuum. If the designer is satisfied with 26 in. of vacuum in the summer, increasing the water rate of the turbine ½ lb. or more, then a 30 to 1 ratio may be used; but if the water rates are to be kept at the guaranteed quantities and the best efficiency secured, then the ratio should be between 60 and 90 to 1, depending on the summer temperature. If the temperature of intake water goes up to 90° Fahr., the ratio should be more than 100 to 1.

A. Streiff,‡ M. Am. Soc. C. E. (by letter).§—Apparently, there is still a great deal of difference among engineers as to how the modern large steam turbine should be supported. The usual specification put out by the Turbine Departments of the Westinghouse and General Electric Companies prescribes that the turbine foundation shall be detached from the building structure. On the other hand, the Allis-Chalmers Company favors the opposite practice of a monolithic building.

---

* This discussion (of the paper by Ernest M. Van Norden and George A. Hughes, Members, Am. Soc. C. E., presented at the meeting of the Power Division, New York, N. Y., January 21, 1926, and published in March, 1927, *Proceedings*), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New York, N. Y.

‡ Cons. Engr., Bangor Hydro-Elec. Co., Jackson, Mich.

§ Received by the Secretary, April 21, 1927.

Detached foundations are made for seismographs and equatorials of astronomical observatories, but to require such a foundation for the steam turbine seems unwarranted. The modern steam turbine is marvelously well balanced, and fear of damage to the structure by reason of undue vibrations seems illusory. The large mass of the building naturally diminishes the amplitude of the vibrations and if these are objectionable, they surely would be excessive for a separate turbine support.

In a shop test on running balance the Allis Chalmers Company's practice is to remove the foundation bolts one by one until the machine is entirely free from its temporary foundations. Such a machine would hardly need to be entirely detached from the remaining building.

The Hell Gate Station shows the turbine supports interconnected with the main structure, whereas in other very large stations as at Kearny, N. J., the turbine supports are separate. The former practice results in a roomy and simple supporting structure and is greatly to be preferred if both systems are equally satisfactory under running conditions.

Again, the requirement of applying a horizontal thrust equal to 25% of the vertical weight on the supporting frames of the turbine, as given by the Westinghouse and General Electric Companies, seems absurd. Such a thrust does not occur under any circumstance. In the direction of the shaft the largest thrust is the one possibly exerted by the steam main, which should be kept small. At right angles to the shaft the largest impact occurs during a short circuit, but this is an internal stress affecting only the supporting beams of the stator frames and does not introduce a horizontal thrust. Such a requirement causes heavy bracing and this interferes more or less with air coolers, condensers, bleeder connections, heaters, etc. The Allis Chalmers Company does not require such a horizontal thrust to be applied to the frame.

The arrangement of the turbine support followed at the Hell Gate Station seems to the writer the correct form. A statement as to its behavior under operating conditions would be of interest.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE RELATION OF HIGHWAY TRANSPORTATION TO THE RAILWAY

Discussion*

By Messrs. John V. Hanna, E. A. Hadley, and F. G. Jonah.

John V. Hanna,† M. Am. Soc. C. E.—It may be said, as a matter of economics, that the traveling and shipping public is going to make use of any new instrument of transportation which offers better service than it has had heretofore. Consequently, when motor cars, buses, and trucks can perform service economically they will enter the field, driving out less attractive or less economical agencies. Efforts to do away with this form of competition, by characterizing it as unfair, and attempting to prevent the issuance of certificates of necessity and convenience, are not likely to meet with success as long as the traveling public shows a preference for such service.

The passenger motor car, it is generally agreed, has caused loss of business to the steam railroads. If it does not always afford cheaper transportation than the railroad, it offers an attractive form of touring, unrestricted by schedules dictated by considerations other than the wishes or convenience of a particular traveler. The motor car has also a distinctly business application, as in the case of the traveling salesman, who can make his trips to suit his convenience, avoiding considerable loss of time waiting for scheduled trains. Many other instances might be cited.

As far as the tourist is concerned, most railroad officers regard the loss from this cause as permanent. The same is true of short business trips. Bus transportation is in a somewhat different category. The bus usually offers to the man who formerly used the railroad train a trip in greater comfort, more frequent schedules, and greater convenience in beginning and ending his

* This discussion (of the paper by Ralph Budd, M. Am. Soc. C. E., presented at the Spring Meeting, Kansas City, Mo., April 14, 1926, and published in May, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Engr., Kansas City Terminal Ry., Kansas City, Mo.

journey. It would appear that this is a field, heretofore occupied by the steam and electric interurban railroads, which such roads might legitimately consider their own and continue to occupy, abandoning the old equipment and route for the new.

It does not appear likely that the bus will take any considerable part of the railroad's long-distance travel. The item of comfort disappears in long journeys, particularly those extending over night, and the frequent stops by buses for local passengers slow up their average speed to a point below that of the express train. There appears to be a logical place for the bus for short daylight runs, and these are more likely than not to parallel the line of the railroad.

There is little doubt that the service now rendered by the accommodation train, traveling 75 to 150 miles, and made up entirely of day coaches, would, for most passengers, be more acceptably rendered by a line of buses with more frequent service, but carrying smaller numbers in each unit. There is a proper use of the bus as a feeder for a line of railroad, also for branch-line service, now given by train. Such cases can only be determined by thorough examination. This has been done by a number of the steam roads and bus service has been inaugurated.

One feature of such service that should appeal to the public is the existence of regulation, both State and National. Bus transportation to-day is frequently in the hands of men with small financial resources. The sentiment that all public transportation agencies or common carriers should be held responsible for their acts of omission and commission has taken firm root. The responsibility of steam railroads for personal injuries, and loss and damage to property is well established, and it seems reasonable that the traveling public should expect and demand the same responsibility on the part of all general transportation agencies. While public demand will undoubtedly call for regulation of bus transportation, whether in the hands of steam railroads or independent agencies, the machinery of regulation of steam railroads is already in existence, is well understood, and can readily be extended to any other means of transportation the railroad may use.

An important consideration in bus and truck transportation, one that appeals particularly to all users of the public roads, is the question of taxes for the construction and maintenance of those roads. Instances have been known of electric interurban lines, the business of which has been seriously injured, if not destroyed, by motor travel on parallel public roads, having been highly taxed for the construction of those same roads. The public road not infrequently has been parallel with and adjacent to the interurban line. The injustice of a tax on the interurban's right of way is apparent and needs no argument. It is not clear, however, that there is an application of this argument to the case of the steam railroad which uses the bus as a feeder, to replace an accommodation train, a self-propelled unit, or in branch-line service. Where train service is replaced by bus service on public roads paralleling the railroads, it may be said that the permanent way is provided for the railroad at public expense in the same manner and to the same extent as it is fur-

nished to other users; and it is equitable that the property of the railroad should be taxed in the same manner as the property of others. There is danger, however, in a direct property tax, such as some States apply, of an assessment out of proportion to the benefit the railroad receives.

The improved country road is often the means of affording readier access to the railroad on the part of the people along its lines who receive their supplies and ship their crops or merchandise over the railroad. Improved roads, offering encouragement to development of the country and the increase of its population, may prove to be a source of revenue to the rail line. To the extent that improved highways promote the growth and prosperity of the country, they promote the prosperity of the railroad serving that country.

Construction of the State Road System of Missouri is being carried out with funds obtained from the sale of bonds, together with portions of the motor vehicle and gasoline taxes. Where the motor vehicle license is proportional, or roughly so, to the horse-power of the vehicle, there is no gross inequity in that license or the gas tax. It is not true that damage to public roads is directly proportional to weight of vehicle and consumption of gasoline; but on the whole there is some approach to fairness in this general proposition, and its simplicity commends it for use in the place of more complicated methods, which attempt to take into account the destructive effects of particular types of vehicles.

The present policy of Missouri is to construct and maintain State roads without a direct property tax or benefit assessment. The motor vehicle and gasoline taxes, it is now estimated by the State Highway Commission, will yield an amount large enough to supply a sinking fund for retirement of road bonds, to pay maintenance costs and, later, to provide money for new construction. By this means it is expected the entire system of State highways, more than 7 000 miles, will be completed without a bond issue other than that of $60 000 000 now authorized.

The use of trucks in making a part of the complete movement of merchandise, from shipper to consignee, has obtained for many years, first, in the shape of the horse-drawn vehicle and, later, the motor truck. The movement between the freight house has not been handled by the rail company until a comparatively recent date and even now is not commonly so handled. The cost of the trucking part of the complete movement has always been paid ultimately by the consignee or consumer as a necessary part of the whole transportation charge. Except for the reluctance of railroad managements to add to their machinery for the transaction of business, there appears to be no sound reason why the rail company should not handle the entire goods movement from origin to destination.

The truck movement, particularly in large cities, has been handled by the shipper or consignee himself, or by trucking companies organized for that purpose, or by individuals. It would be manifestly a convenience, both to the shipper and consignee, if store-door pick-up and delivery were the general practice; the entire movement made under one bill of lading and the collection of charges made by one agency. There should be no increase in total

costs. There might be a slight decrease in cost if all trucking to and from the freight house were performed by one agency instead of a number of unrelated concerns. It is evident, upon a little reflection, that once this trucking unit is organized, its operation can be extended to road haul, as long as the distance traveled is kept within economical limits.

A less than car load movement of, say, 30 miles by rail requires that (1) the merchandise be loaded into a truck; (2) be taken to the freight house; (3) be unloaded and placed in a car; (4) the car switched into a local freight train; (5) be hauled the 30 miles; (6) be set at the freight house or unloaded by the crew of a local freight train; (7) the goods be loaded into a truck; (8) the truck be drawn to the place of business of the consignee; and (9) the goods be unloaded. If this movement were made entirely by truck, six of these operations would be eliminated. The movement in the truck would undoubtedly mean a higher ton-mile cost than the rail haul considered alone; but a point must exist where the total of all costs for the combined rail and truck movement would exceed the total cost of the truck movement. Beyond this point the transfer from truck to car and *vice versa* would be justified by the lower cost of the rail haul. Short of this point the all-truck haul would be cheaper. Without analysis of cost, the length of profitable truck haul cannot be determined—it will vary with local conditions and type of equipment used.

As in the case of buses for passenger movement, convenience to the public is a feature likely to meet favorable reception. There appears to be no good reason why tariffs should not be fixed to give appropriate rates for points within limits of economical truck haul. Again, it is feasible to handle the truck movement by contract with independent agencies, or by the rail company under its own ownership and direction with a separate department. The advantages to the public would be in eliminating some of the agencies with which the shipper and consignee must now deal and in placing the entire movement in the hands of a financially responsible concern subject to regulation by existing State and National bodies.

E. A. Hadley,* M. Am. Soc. C. E.—The author has well stated that transportation always has been a controlling factor in the commercial expansion of a country. One of the highest types of ancient civilization was that of the Roman Empire. Its road construction and maintenance was such that certain of its highways have survived to the present time and have set an example of what constitutes permanent highway construction.

When and how a line of highway should depart from its general course to pass through a small city or town, or to give service to an agricultural or industrial community, is a problem which cannot be settled by any predetermined rule, nor is it likely to be settled to the satisfaction of all parties concerned. The question of deciding locations for highways where only natural physical conditions are involved, causes less trouble to the engineer than where features of ultimate development of the territory must also be considered. A highway located so that it serves as a direct feeder to other transportation lines is a distinct asset to the community as a whole. In the con-

* Chf. Engr., Mo. Pac. R. R., St. Louis, Mo.

struction of such highways the railroad company finds itself most favorably interested.

During the early periods of American history, the Government, by means of Congressional appropriations, from time to time aided in the construction of certain toll roads and canals, as well as inland waterway projects and railroads. These appropriations were spasmodic and followed no definite plan, nor were they equalized among the several States. Probably they were governed largely by immediate military necessity or political influence rather than for any definite purpose of development of the country as a whole.

Norway has records of legislation more than 1 000 years old, providing for the building and upkeep of roads and bridges. In 1274 a rather complete code of highway laws was enacted, which was replaced by a new code in 1604. In 1824 there was another revision and from that time dates Norway's modern road system. Highway legislation in England also dates back many hundreds of years.

In the United States, the interest of the Federal Government began to take definite form when a small department was originated in Washington, D. C., in 1893, called the "Office of Road Survey", the function of which was to collect data on highway conditions. This department was later designated as the "Office of Public Roads", and the scope and volume of its work and its personnel was largely increased until it finally developed into that branch of the Department of Agriculture, now known as the "Bureau of Public Roads."

The Bureau has played an important part in the development of the highway movement, although prior to 1916 it took little active part in local or State affairs, other than in research and investigation work. Largely through its recommendations a provision was inserted in the original Federal Aid Road Act requiring that as a condition prior to receipt of Federal Aid, an adequate State Highway Department must be created and maintained. Beginning with 1916 when the Federal Government appropriated $75 000 000 to be alloted to the States annually over a 5-year period, on a graduated scale, the Government has increased such appropriations from time to time so that the total made for the years 1917 to 1926, inclusive, have amounted to $615 000 000.

It is to be presumed that these funds of the Federal Government have been accumulated by taxation under which the railroad companies have at least paid their proper proportion. As property owners and taxpayers in the several States, they are, therefore, entitled to their proper share in the distribution of Federal Aid to the States in highway construction. Unfortunately, this view does not seem to prevail in certain States, and those in authority fail to consider the railroad company's interests in the application of such Federal aid as is secured.

In other words, Federal aid has been applied to projects which have not involved separation of grades from the tracks of a railroad company, and the cost of such separations has been divided between the State and the railroad, eliminating any consideration of Federal aid funds allotted to the State for

highway work. This is not entirely fair to the railroads. They are setting up the claim that Federal aid should be prorated in such manner that the railroad companies will benefit in the same proportion as other property owners and taxpayers in the community.

The theory of highway taxes for maintenance of roadways is quite ancient. As early as 1661, during the reign of King Charles II in England, four hundred hacks were licensed for operation in London, Westminster, and vicinity, and within a radius of six miles. The revenues from these licenses were used for paving the roadways.

The gasoline tax, as mentioned by Mr. Budd, is an efficient device for making those using the highways pay for such use in somewhat the same manner as toll-gates once required such payments. Thirty-nine States and the District of Columbia now tax gasoline sales, revenue from which is used for the benefit of the highways of those States. According to report the Republic of Mexico has imposed a gasoline tax for the purpose of raising funds for improving highways.

It has been estimated that in 1925 about 20 000 gal. of gasoline were consumed in the United States during every minute. The domestic consumption of gasoline has increased from 4 474 000 000 gal. in 1921 to 8 950 000 000 gal. in 1925, or almost exactly 100% in 5 years. Records indicate that there are in excess of 20 000 000 motor vehicles in the United States, or 1 for every 6 people (which exceeds the number of telephones). Under such conditions the gasoline tax is widely distributed and seems to be the most logical method of raising funds for the construction and maintenance of highways.

A uniform bill for motor vehicle regulation by the several States has been proposed by the American Electrical Railway Association (it also has the support of the American Short Line Railroad Association). The Association has gone so far as to prepare a complete bill which it recommends be passed, vesting authority in the Railroad Commission of the State for the regulation of highway traffic. This bill proposed a tax of 1 cent per ton-mile for pneumatic tired vehicles, and 1½ cents per ton-mile for hard tired vehicles, which figures were arrived at by a committee of the Association after extensive studies. The Committee admits that the information on which the amount of the taxes was based was incomplete, but believes that any change which may be necessary in the future will be in the form of an upward, rather than a downward, revision.

The railway companies have complained because of the general and special taxes they are obliged to pay to help construct and maintain highways, on which motor vehicles compete with them. Spokesmen of the motor industry have replied by showing that the total taxes paid on motor vehicles are larger than on railroads and by referring to the large amount of freight business that the motor manufacturing industries give the railways.

The fact, however, must not be lost sight of that part of the taxes collected from the railways are used to pay for the highways on which motor vehicles operate, whereas none of the taxes paid on motor vehicles is used to help provide highways for the railroad trains. Many taxes assessed against

motor vehicles are against private passenger automobiles; therefore, the total amount paid is no indication as to whether or not the taxes now being levied on motor buses and trucks are adequate. The railways operate on roadways which they themselves construct and maintain, while buses and trucks operate over highways which the public have constructed and maintains. Eventually, they must pay in some form their *pro rata* of this cost if an equal balance is to prevail.

Regulation of buses and trucks operating as common carriers appears to have the unanimous approval of virtually all interests. The principle of taxation has similar approval; however, the amount of the taxes which would be fair and just is a point on which there is sharp controversy. It appears to be generally agreed that taxation should not be adopted for the purpose of strangling highway transportation, and the conservative elements on both sides of the question advocate a tax schedule that would impose a fair share of the cost of construction and maintenance of highways on trucks and buses without unduly burdening them or restricting the development of this form of transportation.

F. G. JONAH,* M. AM. SOC. C. E.—Transportation is one of the distinguishing characteristics of civilization. Savages and barbarians had none, or, at best, very crude methods. Primitive man traveled very little. The first migrations that he made were probably in following game and for spoilation or conquest of neighboring tribes. In the earliest forms of transportation man himself was the vehicle, burdens being carried by women and slaves. After a time he learned to domesticate and harness animals, and thereupon transferred the burden of transportation from his own shoulders to the backs of four-footed creatures.

All transportation is founded on the use of the wheel, also much of the industrial development. At a very early age when the wheel was invented, transportation took an immeasurable stride forward. The first development of the vehicle on wheels was the chariot of the ancients, harnessed to horses, and used largely in military operations. The necessity of furnishing some path on which these vehicles could travel gave birth to the good roads idea.

The Romans built the great roads of antiquity as a basis for military, rather than commercial, operations. However, they substantially formed the basis of the vast network of highways which became of great use in the development of Central Europe.

There was little change in transportation methods for a period of almost 2 000 years, the only difference being that vehicles became more ornate, more commodious and more substantial, and that the roads on which they traveled became better. It was not until 100 years ago that anything was evolved that could be termed "rapid transportation" and that possibly only after mechanical means were utilized for propulsion.

In 1925 England celebrated the one-hundredth anniversary of the founding of the steam railway. Railway development in the United States began about 90 years ago. It was far more rapid, efficient, and economical

* Chf. Engr., St. L. & S. F. Ry., St. Louis, Mo.

than anything the world had previously enjoyed, so it is not surprising that on its introduction all other forms of transportation quickly fell into disuse. The stage coach, pony express, river steamer, and canal boat became almost obsolete in the United States, and the fact that railway transportation was developed so quickly and so efficiently led for a time to the neglect of the highways. In this respect America was entirely different from the European countries because their highways had been established and well developed before the steam locomotive was invented.

About forty years ago electricity was introduced as a motive power for transportation. At once the interurban car became exceedingly popular for the simple reason that it could traverse the streets of cities and villages, could be boarded at almost any street corner, and made frequent trips between various parts of the cities and adjacent country districts. Then the railroad companies were faced with a new competitive factor, one with which they could not wholly compete for short-line hauls.

A few years later, or twenty-five years ago, the automobile became a reality, and its use, with the motor trucks, has become so general that an entirely new situation has developed in transportation. About the first important result of the widespread ownership of automobiles was an insistent demand for the construction of good roads, and the various States undertook to supply this deficiency with characteristic American haste and waste.

Good roads have been, and are now being, developed along previously established routes of communication; that is, generally they are parallel with the trunk lines of the railway companies. It is not at all improbable that many expenditures thus made were unnecessary, and that more good would result to the country eventually if the roads had been built into remote districts, making them accessible and tributary to the existing railways.

The centers of population were built along the railroad lines, which usually represented the shortest distance from one flourishing community to the next. As the object of the highway systems was to link up important towns, it was natural that the railroads would be paralleled to a great extent.

On the completion of these highways, bus lines were introduced, in close competition with the railways—in some instances, with the electric lines. Being more flexible and mobile in their movements, they have become a highly competitive instrument in transportation, so that in many cases railway companies have been obliged to curtail their local passenger service. To the present time they have not entered the field of freight carrying to any great extent, although that is coming. Certain companies in the East have put on freight-carrying buses to handle their local freight traffic within certain territories. This has one distinct advantage—it makes store-door delivery possible within a certain radius.

In the early days of railroad development it was regarded as very essential to have freight and passenger stations as near the heart of the city as possible. This was in the days of the horse-drawn vehicle when the haul was of particular moment; but nowadays, with well-paved streets and quick-moving motor vehicles, it is not so essential, and cheaper sites on the out-

skirts of the cities can be obtained. The moving of these freight terminals probably will lessen congestion on the business streets of the city to a great extent, so that eventually the railroad companies may be able to make substantial savings. In many instances the railroad companies own extremely valuable properties in the hearts of cities, which may be dismantled and the ground sold for more than enough to erect modern and commodious terminals on the outskirts.

The question arises as to what is the railroad company to do to meet the bus competition? To some extent it can, and does, put motor-driven cars on its rails, as for local passenger service; but this does not fully meet the situation because the car is still confined to the rails, and is not as mobile as the bus, which can gather up its passengers around the street corners of the cities.

As to less than carload freight, the time is undoubtedly coming when manufacturers in cities will distribute goods from their factory doors to their customers within a radius of many miles by their own motor trucks; or the service may be performed by truck companies, the goods being only handled twice—once from the factory into the truck, and again from the truck to the store. Evidently, there is some economy in this handling as compared with methods pursued in the past. Just as the railroad companies performed a more efficient service than had hitherto been enjoyed, so the motor bus and motor truck, with the possibility of cheaper transportation, will undoubtedly secure a part of the business, and the railroad companies will have to adjust their operations accordingly.

There is a widespread movement for American railroads to enter into motor-bus and motor-truck transportation.* Railroad companies are applying to the Interstate Commerce Commission for permission to publish joint through-passenger fares covering transportation partly by train and partly by motor bus. Several companies are adding greatly to the buses already in service.

The Chicago and Alton Railroad Company announces that in the near future it will put on motor buses to travel the highways between certain important points on its lines. On no other railroad in the Middle West could this experiment be tried with as good prospects of success, nor would the experiment be so well justified. Situated as it is across the populous State of Illinois, extending from Chicago to St. Louis, Mo., and having many large and important towns only a few miles apart over its entire distance, and paralleled practically the entire way with an electric line on one side and a concrete highway on the other, this railroad seems to be ideally fitted to try out a system of motor transportation under railroad management. One result of the developments of the last few years has been that local passenger trains have become much less generally used, and the chances are that in the next few years many local freight trains also will disappear.

* *The Railway Age*, March 27, 1926.

The good roads have been and are being established, and the motor bus and motor truck are being more efficiently developed from year to year. All three are fixed parts of the transportation plan, and the railroad company will have to utilize these and add them to its established facilities as another arm of its service.

It is not at all unlikely that the railroad companies, with their skilled shop forces, whereby repairs to motor equipment can be maintained more cheaply than by private companies, will be in a position to render this service perhaps cheaper than any other agency. It can be set down as a certainty that the cheapest and quickest method of transportation will, in the end, be the most used. There is no possibility that for long hauls any method can supplant the railroad for the movement of either passengers or freight.

The increasing use of the automobile, motor bus, and motor truck is having a pronounced effect on rural life. The growth of small country towns as business centers has been arrested. The country store, country school, country church, country doctor, and country newspaper are passing, and business activities and population are being centralized. The local business of the railroad companies is being affected by this tendency.

Years ago, when railways were first built, and in the days of horse-drawn vehicles when highways were generally in bad condition, it was thought necessary to have stations about seven miles apart. Now, when communication is so much more rapid and easy, business will seek better markets, and there is a strong likelihood that in many instances the local stations of the railways can be abandoned, and possibly the railway companies may be able to affect some savings by reducing service where it is no longer needed.

The railroad companies have not always been treated fairly by State Commissions in respect to the curtailment of their passenger service. It has been clearly shown in many hearings that the introduction of bus lines has taken away the necessity for the operation of local passenger trains and yet the Commissions refuse to allow the companies to change their schedules. The time is at hand when a comprehensive plan of regulation must be worked out, governing the operation of buses and trucks engaged for hire on the public highways; and the interests of the railroad companies must be taken into account by the regulatory bodies. The aim should be adequate service without wasteful duplication.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

URBAN AND INTERURBAN BUSES

Discussion*

By Messrs. Robert P. Woods, Fred G. Buffe, B. H. Piepmeier, C. E. Smith, Winters Haydock, John W. Reid, and Lucius S. Storrs.

Robert P. Woods,† M. Am. Soc. C. E.—Motor coach operation between cities and towns as a unit of transportation has grown rapidly since 1924. Included in this interurban service are the necessary and the unnecessary, the desirable and the undesirable. Vehicles ranging from the luxurious 30-passenger, high-powered, $12 000 coach to the well-worn touring car are in use. Here and there the business is standardized to a certain extent, but, on the whole, the situation, due to the quick growth and misapplication to existing necessities, is far from being stabilized. The character of equipment, amount of service, rates of fare, and ownership of lines and companies are changing rapidly and new routes are opening. The lines of the routes vary from a few miles to several hundred, and the rates of fare from 1.5 to 5 cents or more per mile. The higher rates apply more particularly where infrequent service with light patronage is given. The average rate is low. The schedules range from one or two trips per day to regular hourly and half-hourly service.

Thirty-eight States have passed laws controlling interurban coach service, but not Missouri. A number of the States made changes to their laws at the sessions of 1925-26. An intensive move is under way for interstate regulation by the Federal Government.

The motor coach service is welcome and popular with many people. Helpful in this direction has been the development and great use of the automobile, of which there are now 20 000 000 in the United States, a sufficient number to enable practically the entire population to ride at the same time. Few,

* This discussion (of the paper by Britton I. Budd, Esq., presented at the Spring Meeting, Kansas City, Mo., April 14, 1926, and published in May, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Vice-Pres. and Gen. Mgr., Kansas City, Clay County & St. Joseph Ry., Kansas City, Mo.

therefore, are immune to its pleasures, its readiness to serve, its speed, and its accessibility. The number justifies the statement that the people have formed the riding habit.

Another help has been the extension of improved hard-surfaced roads. Motor coach manufacturers have developed a high grade equipment and this, in contrast with the present character of many interurban electric railways, has been favorable to the motor coach patronage. Interurban electric railways, however, generally have been handicapped financially because they suffered a loss in net earnings, due to the low rates of fare and the increased operating expenses. Motor coach carriers may encounter the same handicap. The tremendous use of the private automobile has created a restless spirit that will brook no delays, and it is to this more than to any other factor that the growth of motor coaches may be attributed.

The motor coach as a carrier is expensive in initial cost, operation, and maintenance, more so than the public realizes—in fact, more so than many of the newer operators believe, because the maintenance and renewal expenses will come as a future revelation.

The cost of owning, operating, and maintaining a private automobile is no small item, but when a person rides or drives in the family car the feeling of pleasure and personal interest discounts the expenses, and if a similar spirit is felt in riding in the public motor coaches, sufficient revenues may be expected to sustain good service. It should be remembered, however, that the present status as a carrier has been built up during a period of low fares.

Motor coaches are in operation where formerly there was no transportation. They are used as feeders to steam and electric railways, and for special trips, both long and short; also, where the service was deemed inadequate, as along steam railroads with only one round-trip train each day, the stations of which are somewhat remote from the center of the retail business districts.

Opinions as to the future of interurban motor coach transportation will be varied, depending on familiarity, knowledge, experience, and personal interest. In part, at least, it would hinge upon "whose ox is gored". Among those looking at the subject from this angle will be the coach manufacturer, the coach owner, steam and electric railway officials, the coach salesman, the accessories representative, those receiving special benefits from its operation, and, lastly, the public.

The speaker will try to be as free of bias as possible. As an independent unit of transportation in the interurban field the success of the motor coach as a carrier will be spotted. In progressing it will meet the same measure of regulation as that given to the regular transportation carriers as soon as it emerges from the swaddling clothes of newness. If the volume of traffic becomes heavy, it, like the electric interurban roads, will probably be forced to secure private right of way in such sections. This the coach line would require for safety and to maintain its speed; and the authorities would require it on account of traffic congestion and extra wear and tear on highways. The steam and electric railways are getting into the field more and more and, in time, will probably control the greater number of parallel coach lines.

Much is said at present of the "co-ordination" of coaches with other carriers, but the word seems not to have the meaning given in the dictionary. The intent in many places is rather to parallel and "cut under" the regular carriers. Proper co-ordination, however, is needful for the public good. In time, it will prevail, but when it does the operation of the motor coaches will be on a different basis from that prevailing to-day. Duplication of service is an economic loss, first, underwritten by the interests involved, and, later, paid for by the public, whose portion is not always earmarked, but who finally settles for it in one way or another. Excess service, especially when unprofitable, produces bad after effects; the equipment deteriorates and then recovery brings higher rates for the public to pay—in short, the public pays "going and coming".

It must not be inferred that the interurban motor coach carrier will be banished—far from it; it will fit into the transportation picture nicely, but not as operated at present. Its development has been remarkable and in these days of enterprise and keen competition it may be that the industry could not have attained the wide recognition it has by any other introductory means than have been used. The justice of the means in some instances, however, is at least questionable.

A tabulation of the various coach lines throughout the country, showing the complete character of service, local conditions, volume of traffic, competition, and other pertinent facts, would disclose interesting information from which a reliable estimate, rather than a guess, could be made of the future tendency of the industry; but such a summary would be voluminous and hardly worth the effort, for before it could be compiled and studied the situation would be changed. However, a short review of the interurban motor coach service in and out of Kansas City, Mo., will supply definite knowledge that is applicable to the conditions found in a large part of the industry as a whole.

Interurban motor coaches are operated between Kansas City and seventeen other terminals. Twenty separate coach lines are operated and competition extends over three, and a part of a fourth, of these seventeen routes. Three of the routes practically parallel electric railways and all cover territory served by steam railroads. The total length of highways traversed (not doubling where competing lines exist) is 1 773 lin. miles. The twenty coach lines run 250 total trips daily into and out of Kansas City, covering, all told, 12 419 coach-miles, or a distance equal to half way around the earth. On sixteen of the twenty coach lines the vehicles seat twelve or more persons, and, on the remaining four lines, touring cars are used. These various routes with their length are given in Table 1.

The rates of fare vary from 1 to 0.7 cent per coach-mile, as in the case of the two lines to Denver, to 3.9 cents per mile. The rate from Kansas City to St. Louis, on both lines, is 2.6 cents per mile on a single-trip basis, or 2.3 cents on a round-trip basis. The numerical average rate of fare for all routes is 3 cents per mile one way and 2.9 cents per mile on a round-trip basis.

The net earnings of these lines are not known, but as they are dependent on volume of business, kind and amount of service given, depreciation provi-

sions, and rates of fare, it is believed that, taken as a whole, there has been no profit. Recently,* much to the surprise of many, seven or eight of the large interurban motor coach lines operating out from Indianapolis, Ind., have failed. Time is bringing adjustments to the coach industry throughout the country, and the Kansas City field will show modifications from time to time due to the foregoing factors and such regulation as may be added by legislation.

TABLE 1.—VARIOUS MOTOR COACH ROUTES AND THEIR DISTANCES INTO AND OUT OF KANSAS CITY, MISSOURI.

Place.	Miles.	Place.	Miles.
Topeka, Kans.	75	Higginsville, Mo.	58
Leavenworth, Kans.	33	Sweet Springs, Mo.	62
Excelsior Springs, Mo	32*	Warrensburg, Mo.	60
St. Joseph, Mo.	60†	Harrisonville, Mo.	41
Parkville, Mo.	10	Lees Summit, Mo.	20
Plattsburg, Mo.	38	Osawatomie, Mo.	62
Lexington, Mo.	47	Denver, Colo.	677*
Booneville, Mo.	104	St. Louis, Mo.	267*
Columbia, Mo.	130		

* Two separate companies.
† Part-way competition.

An example of operation in competition with long-established steam and electric transportation is the line along the Kansas City, Clay County and St. Joseph Railway. This railway, situated entirely in Missouri and one of the outstanding interurban electric railways of America, has been in continuous operation since early in 1913. It is a high-speed, heavy-type, electric road, with a 1 200-volt catenary trolley, all steel passenger cars, automatic block signals, and all on private right of way, with a dozen or more grade separations. The train service is hourly, with half-hourly service at rush periods. Many of the trains are of two cars. The rate of fare is 2.5 cents per mile.

On February 15, 1924, the new, hard-surfaced highway between Kansas City and Excelsior Springs was opened to traffic, at which time a motor coach company (Yellow Line) began operation between the two terminals, paralleling the Electric Railway with large high-type motor coaches on a 2-hour service, which by June 1 was increased to hourly service. At rush-hour periods it was necessary for the company to send out two coaches seating twenty-six persons each. The Yellow Line is still operating and as a competitor of the Electric Railway.

On August 7, 1924, the Kansas City, Clay County and St. Joseph Auto Transit Company (Blue Line) owned by stockholders of the Electric Railway and managed by the latter's officers and heads of departments, inaugurated a 2-hour service between Kansas City and Excelsior Springs, with large de luxe motor coaches, thus paralleling the Electric Railway and the Yellow Line. This service being too infrequent to draw patronage, it was changed to hourly service, with extras during the rush-hour movement. The combined heavy

* Between March and April, 1926.

service of both coach companies, amounting much of the time to about ninety coach trips per day, continued until May, 1925. During this period the passenger cars of Electric Railway showed a heavy decrease of patronage, yet it maintained and still maintains its former schedule of hourly service, with a half-hourly rush-period schedule. Beginning with May, 1925, by agreement, each coach company reduced its service to an hourly schedule, thereby eliminating the extra service that had developed.

Three reasons may be given for the attitude of the Electric Railway Company: One was a spirit to give to the public coach service to the extent it desired; another was to protect ultimately the $5 125 000 invested in the property; and the other was to indicate to those contemplating further paralleling of the line that they would have competition in kind. For considerably more than a year the Blue Coach Line (ally of the Electric Railway Company) has operated from Kansas City to St. Joseph, 60 miles, and from Kansas City to Parkville, 10 miles. It has no bus competition between the terminals at Kansas City and St. Joseph. The line to Parkville has no competition; it partly parallels the Electric Railway, but also acts as a feeder.

Notwithstanding the frequent and high-grade motor-coach service given by the two companies between Kansas City and Excelsior Springs, patronage has been steadily falling off recently (1925-26) on both and that on the Electric Railway has been increasing. Previously, the combined coach service was twice that of the Electric Railway. From March 31 to April 6, 1926, there was no motor-coach service on the coach lines, due to a heavy fall of snow. Upon resumption, the Yellow and Blue Lines, on account of the falling off in business, agreed to operate coaches every two hours instead of hourly, with some modifications, such as an extra coach for the short haul at 6:00 P. M. and the partial elimination of the last or theatre run. A few figures will show the change in the flow of patronage from the coaches to the Electric Railway between Kansas City and Excelsior Springs.

During the calendar year 1925 the railway, with about one-third of the combined service of the Electric Railway and two coach lines, carried twice as many passengers as both the coach lines and during the last five months of 1925 it carried two and one-third times as many, showing the increasing tendency to switch to the Electric Railway.

For the first three months of 1926 the number of coach passengers on both coach lines has decreased about 32%, and the number of passengers on the Electric Railway has increased about 33%, to a number 2.94 times as large as both motor coach lines. During this period the coaches curtailed their mileage, while that of the Electric Railway was increased, the railway supplying 38% of the total mileage.

Therefore, with rates of fare practically the same on all lines, the combined motor-coach patronage with its extensive service decreased, as compared with the year previous, with the rate of decrease greater during the last three months. Whether or not this tendency will hold is problematical. The motor coach is here to stay. Adjustments and modifications will be made from time to time and gradually there will be a real co-ordination with the other long-established and more reliable transportation carriers.

FRED G. BUFFE,* ESQ.—It seems indicative of a better era in business to find thinking men actively engaged in constructive work in their communities, seriously studying transportation, and by so doing helping those in the business to solve their problems.

The bus has been, and still is, a problem. To the unthinking it may not seem so. It is easy to say, "We like this new method of riding—the smell of gasoline and a ride on rubber—and if the street railways, the interurbans, and the railroads will not furnish such rides, three cheers for the fellow who will." Such was the attitude that existed when competing street railways were built; when competing railroads were invited; when gas and electric franchises were had for the asking, on the theory that competition is the life of trade, and "the more the merrier." It took many years of loss and efficiency before this mistake was generally recognized and before the public accepted *in toto* the theory that certain services, chief of which is transportation, are natural monopolies, and that regulation and not competition is the proper check against abuse.

Therefore, the bus must be considered not only as a vehicle capable of carrying people, but in its relation to the transportation situation as a whole. In the last analysis the problem and its proper solution becomes that of the public. There is danger of magnifying the importance of the motor coach and of giving it a disproportionate place in transportation. This is natural with a new development, especially one that has enjoyed the advertising and propaganda given to the bus. People are prone to like new toys.

A bus system is a valuable agency in urban transportation when it can be built carefully and properly, with due regard to other facilities. If, because of too much propaganda or threatened competition, or without due regard to its effect on transportation, it is used improperly, it will in the end work a real public harm where there is an apparent public gain. It is valuable because of its flexibility. It can be used anywhere—routes can be changed, detours made, and delays avoided. It requires a lower investment per seat furnished. It affords an easy and less costly method of making extensions, of serving undeveloped territory, of instituting cross-town lines. It readily lends itself to park and boulevard service that have grown away from car lines. It fills a gap between the street car and the taxicab, and gives the street railway operator a vehicle that appeals to his former patrons who have deserted his service for the private automobile.

The bus is not, and very evidently never can be, a substitute for the electric railway in moving large numbers of persons during the rush hours. The street railway is here to stay, and its usefulness will increase as the vehicle for mass transportation at the lowest possible fare. The street car is more reliable; its rides can be furnished at less cost; it takes up less pavement space per passenger moved; it moves on a fixed track and congests traffic the least, considering the number carried. The street car is the backbone of urban transportation, the great servant of the masses, the foundation on which rests the

* Vice-Pres. in Chg. of Operations, Kansas City Public Service Co., Kansas City, Mo.

superstructure of real estate and business prosperity, furnishing the maximum service for the minimum fare. The bus is primarily a "seat-for-every-passenger" vehicle offering a preferential service at a higher fare.

As confirmation of this statement, it is only necessary to call attention to the present-day electric railway activity. There is a great awakening in every city; new equipment is being bought; tracks are being rebuilt; higher standards of service are being maintained; the more important street railway securities are no longer a drug in financial centers. These things are not by chance. Bankers of long experience would not advise or permit such programs if these properties were going out of business over night.

The competing bus is a pirate in its very nature. Instead of rendering a public service as it seems, it is in reality working a great public injury. It is a backward step in urban transportation, and at some future date the bill must be paid. It will be paid in lack of capital for improvements, in impaired service—often in forced consolidation by which the promoters collect their profit and the street railway as a public agent pays the loss, which, in turn, becomes that of the public. Such competition exists only because it is permitted to serve a preferred territory having great traffic density and a high load factor. It does not assume the burden and the obligation of city-wide service, of undeveloped lines and districts, but leaves these to the street railway while it takes the business the railway must have to support its service obligations. With its system of flat fares and universal transfers, city-wide transportation is properly a municipal problem, and certain sections can be served only because the better lines support the entire structure. Unwise bus competition has often been most unfair, in not being given the advantage of short hauls and profitable territory, but rather in being relieved of paving, street cleaning, and tax burdens, with which its older competitor has been laden.

The proper place for the bus is as an integral part of the co-ordinated transportation plan. Webster says, "to co-ordinate" is "to adjust, to harmonize". With this adjustment, this harmony, transportation may be developed along proper and natural lines. Co-ordinated transportation contemplates a complete service, using both bus and street car, not making one subordinate to the other, but fitting each vehicle to its proper field to avoid duplication and to render a more efficient and desirable public service.

The bus when used as a feeder, as a server of undeveloped territories, such as cross-town facilities and on lines where track must be rebuilt and where there is no rush-hour problem, plainly meets a real need. It can also be used as a main or through-line carrier, serving districts heavily built up away from car lines and along boulevards and parks where car lines would not be permitted.

On the other hand the bus should not be used half-heartedly and confined to the outskirts. The public desires bus transportation, and transportation agencies must meet this public desire. The down-town bus can be used to advantage. It offers a different service, makes an appeal to a certain class of riders who are willing to pay a higher fare on a "seat-for-every-passenger" basis. There is also in many cities opportunity for a limited stop, express service from distant residential districts at a 25-cent fare.

Kansas City, Mo., operates 69 buses on four down-town routes, one express route, one cross-town route, and four feeder routes, a total of 44 route-miles. The fare is 10 cents without transfer except on feeder and cross-town routes, where transfers are given from bus to street car and from street car to bus on payment of a 3-cent differential. The express fare is 25 cents. In all, 200 000 bus-miles per month are run and 540 000 passengers carried.

This bus installation is a good example of co-ordination and of a proper public attitude. Kansas City desired buses. There was an extended investigation by merchants' associations and others, and the general public opinion was that such service should be available. A franchise was submitted by promoters financially able to carry out their commitments. Had this competition been successful in Kansas City, it would have wrecked The Kansas City Railways and postponed the lifting of the receivership indefinitely. Incidentally, it may be said that jitney competition in 1918 and 1919, taking $3 000 per day in revenue, was a large contributing factor to the receivership.

The public, the press, and the city authorities took the position that in providing the desired bus service, the first opportunity should be given The Railways, and that competition should not be permitted. As a result, after careful analysis and planning, The Railways have co-ordinated bus transportation so as to furnish a maximum of service and not compete directly with existing facilities.

The bus, properly developed and co-ordinated, has a future as a transportation agency. It will be further perfected and refined. New uses for it will be found. It will develop its own clientele. It will do its part to solve the traffic and parking problems. It offers new opportunities to serve as well as new problems to solve, and in the end its proper development will mean a distinct public gain.

B. H. Piepmeier,* M. Am. Soc. C. E.—Urban and interurban transportation in its various stages of development has been confronting civilization since the beginning of time. The rapid development of different kinds of transportation has been watched with interest. Many have predicted that the truck and bus could not compete with the electrical car and train, but since their inception, the truck and urban and interurban buses have continued to increase in size, speed, and number. The growth of this form of transportation shows that it is increasing in public favor.

The flexibility and convenience of motor-bus transportation have made it very popular. In the congested centers it is being more extensively used, as it provides a means of transportation at a faster rate of speed and with less congestion than perhaps any other form that has been developed to date. The business man who in the past has been driving his private car to the office and parking it at or near his place of business, is now forced off the street by congestion and parking regulations, and has substituted the use of urban buses.

Every one will concede that the motor car and bus have afforded a means for rapid growth and development of suburban communities. It would be

* Care, Diversified Investments, Incorporated, Kansas City, Mo.

difficult for any one, at this time, to predict the future development of this form of transportation, or just what effect it may have on the future growth of suburban communities. At present, in nearly all parts of the United States, urban and interurban bus traffic is developing by leaps and bounds. During the last five years the vehicles themselves have become almost the last word in convenience and pleasure. Some may raise the question that they are developing faster than they should, or beyond the point of economic transportation.

The maintenance of the road-bed for both steam and electric railroads has been one of the chief expenses imposed on transportation companies, and the public is just beginning to realize that it is shouldering this tremendous upkeep due to urban and interurban bus transportation. It is perhaps impossible, at this time, to estimate just how much of this expense should be assigned to the companies. Modern highways and streets are being designed to withstand the present-day motor bus. It is evident, however, that roads built ten years ago were not, as a general rule, designed to meet the requirements of modern use. One of the big problems, therefore, confronting the engineer and business man of to-day, is to work out an equitable tax which should be charged against urban and interurban buses, so that they may pay their share toward road construction and maintenance costs.

If the public were entirely dependent on buses for transportation, the question of taxing commercial vehicles would not be so serious; but public highways are designed, built, and maintained primarily for individual use. The commercial use, therefore, presents a problem of taxation that has not yet been solved. The public should not pay an indirect obligation from which it will receive no benefit, and yet the bus should be able to continue to serve and aid in the development of community life.

It is a matter of experience that all roads wear out or deteriorate with use, thereby impairing their function in the transportation system. To overcome this, constant maintenance must be provided and replacements must be made at intervals. The traffic is responsible for the deterioration, and, hence, should pay sufficient fees and taxes to meet the annual cost of maintenance and of periodic replacements. In Missouri, all motor vehicles pay a part of this obligation through license fees, based on the horse-power of the engine, and through a gasoline tax, based on the number of gallons used. There is, however, another condition peculiar to motor buses operating for profit—they are using a roadway built at the expense of the motor-vehicle owners at large, and are making a profit out of such use; they are getting a benefit from investments in roadways out of proportion to their contribution to such investments. In addition to the regular license fees and gas tax, they should pay such proportion of the interest charge as the motor-bus traffic bears to the total traffic.

To illustrate this principle, consider as an assumed case—a road 20 miles long, costing $35 000 per mile, and carrying 1 500 tons of traffic per day; over this road motor buses having a gross weight of 4 tons make 14 trips per day, or an equivalent of 56 tons of traffic. The motor buses, therefore, pro-

duce $\frac{56}{1\,500}$ of the traffic, and should pay interest at $4\frac{1}{2}\%$ on $\frac{56}{1\,500}$ of the cost of constructing the road, or about $1 200 per annum.

Motor buses may be able to pay such fees if properly regulated. If they are allowed to operate promiscuously on the streets or highways, they may cause unnecessary damage and encounter such competition that they will not be able to pay their proportionate part of the construction and maintenance expenses. All motor buses, therefore, should be granted permits on the basis of a public necessity and be regulated and controlled by public service commissions.

Engineers should also give more time and study to the design of highways with reference to the economical movement of traffic. Railroads and electric lines have, in general, received much more study than the highway. The outstanding reason is that they have developed gradually over a period of years and this has given engineers a chance to study their needs and to establish economic principles, whereas the motor vehicle has developed so rapidly that engineers have been forced to overlook, in many instances, the economic location and design of roads, in order to keep traffic out of the mud.

Much has been done during the past few years in developing a type of pavement that more nearly meets the requirements of modern transportation. The heavy bus and the speed with which it is driven over the highways has done untold damage. This condition, however, is no cause for complaint against the bus or the engineer who has been directing the work of building roads. The bus has developed so rapidly that time or money would not permit the building or rebuilding of roads to meet the new requirements. It is encouraging to know, however, that the newer types of roads will withstand this modern type of transportation.

There must be developed, however, a sentiment for more uniform regulation of all modern traffic to insure economy in the design of pavements and the proper development of urban and interurban buses. The carelessness of the operator, in many instances, has developed a sentiment against the bus. While it is generally admitted that this form of transportation will continue to develop, it is also agreed that it must be regulated and properly controlled to insure its greatest usefulness.

The question of additional width of highways should be considered very seriously, especially since buses travel at a rate of 45 to 50 miles per hour. It is apparent that either the buses must be narrowed; the main roads, widened; or additional routes, built to meet all the requirements of modern traffic.

The answer is to build additional routes in the suburban territory rather than wider routes. The highway between Kansas City and St. Louis, which is known as the air-line route, is 26 miles shorter than the shortest railroad between these two points; it is, however, only 18 ft. in width on a 60-ft. right of way. When the system is completed these two main terminal cities will be served by two or three other routes across the State, which will serve not only the two main terminals but all intermediate points. It is very evident, therefore, that additional routes render greater service to the public than the

building of wider roads or super-highways. This does not generally apply to congested districts such as exist around Kansas City, St. Louis, and Detroit.

C. E. SMITH,* M. AM. SOC. C. E.—The extent to which the local urban and interurban motor-bus and electric-railway situation has been misunderstood is well illustrated by the statement of L. F. Loree, M. Am. Soc. C. E., before the St. Louis Chamber of Commerce to the effect that an investment of about $6 000 000 000 in street and electric railways in the United States should be written off the books because the automobile has thrown these systems into the discard just as clearly as the steam railroad has superseded the stage coach of olden days. His words created quite a sensation; but few transportation experts will agree with him. They are true of some street railway tracks in the congested centers of large cities where the transportation is furnished by rapid transit lines on upper and lower levels and of many interurban lines; but the street railway will remain the backbone of the transportation system in all other large cities.

There is a big field for the motor bus. Its principal use will be for service beyond the ends of steam railroad and interurban lines into territories where it has not been economical in the past, and would not be in the future, to build and operate rail lines. This is particularly true of the suburban districts surrounding large cities.

In the past the radii of residential areas were pretty well-defined and limited by rail lines because there was no other means of mass transportation; but with the coming of the automobile those limits have been extended. With cities spread over much larger areas the density of population has been reduced. Whereas railway lines have been serving districts in which the population has been more dense, outlying residence areas to-day have from 2 to 6 people per acre. Yet these people need transportation. It will not pay to build rail lines for them, because the density of population will not support such lines.

In city service, there is need for motor-bus transportation only in coordination with established rail routes; in competition it would mean duplication of service, costing more in the long run. It is uneconomical to operate cars and trailers during the crowded hour, carrying 250 people on each train, for local service in an intermediate area; each time the car and trailer stop for one passenger the whole train is delayed. This has been necessary in the past because it has not been feasible to provide both local and express service on street car lines but now it is possible to supplement the rail service by motor buses which will take care of the local traffic—the street to street traffic within the intermediate area—while the street cars run express, stopping only at cross-town streets.

On the whole street car traffic appears to give indications that it is "coming back"; as a matter of fact it never got very far away except in particular instances. Statistics issued by the American Electric Railway Association show that street cars to-day, and for several years past, have carried about

* Cons. Engr. (C. E. Smith & Co.), St. Louis, Mo.

17 000 000 000 cash-fare passengers each year. That has not increased very much, but it has not decreased to any appreciable extent. Automobiles and motor buses have taken away the normal increase that would otherwise have used the rail lines. The future is yet uncertain. In Greater New York the rapid transit lines carry about 1 500 000 000 cash passengers per year, the street cars, about 1 000 000 000, and the motor buses only about 100 000 000.

St. Louis, Mo., gives a good example of competitive systems. Its large street railway system is laid out with radial and circumferential lines on good engineering principles. There is also a motor bus system that the promoters tried to lay out as a radial system with cross-town lines, to compete with the street railways. The street cars carry 90% of the riding public, the motor buses, 10 per cent. First the buses were put on the principal streets and took the "cream" of the traffic. They were quite profitable. Later the bus company secured permits to operate on additional streets into territory where the street car company could not afford to put lines. With this additional service the motor buses are not paying, although the fare is 10 cents. The car fare is 7 cents, but by reason of motor-bus competition and diversion of business to automobiles the car company is taking steps to increase its fares. It is paying about 11% of its gross revenues in all sorts of public charges; the motor-bus company pays about 4%, which does not reimburse the city for the pavements the motor buses wear out. The buses do not carry their share of the peak load. The peak load on the street railway is six to eight times the base load during the day, whereas the buses provide very little more service during the peak than during the day, when there are sufficient empty seats on the cars to carry all the passengers using the buses.

The failure of the motor buses to provide peak-load service increases the peak load on the street railway line. In bad weather the uncovered double-deck motor buses have their loads cut in two, the half lost being thrown on the street railway—a bad condition during rush hours. This is being corrected gradually by the enclosure of the upper decks of the motor buses.

Street railway fares in St. Louis amount to about $1 500 000 per month. If it were possible to take off all street cars and put on enough motor buses to carry the whole load, the transportation bill would increase another $1 000 000 per month. It may be that the psychology of gas and rubber has led to the point where the public is willing to pay that much more for motor-bus transportation, but it does seem that eventually when money is not so plentiful people will look a little closer at this material difference. The same relation applies to other large cities.

One reason why the public has welcomed competitive motor-bus transportation even at higher cost, is the dissatisfaction with street railway service and strap hanging. People have been educated by the automobile to get around more quickly. All other means of transportation have improved, while the street car, which, in St. Louis carries nearly 1 500 000 cash and transfer passengers per day—about 75% of the traveling public—does not move any faster than it did before.

Motor buses performing local service in large cities have proved a disappointment as far as speed of transit is concerned, but many people prefer them, when standing passengers are not carried and for other reasons, real or fancied. In large cities, street cars run at an average speed of about 10 miles per hour. The motor buses run about 1 mile per hour slower than the street cars in St. Louis; unless some means are found for speeding up street car and motor-bus service, people will find ways of getting around faster, and the value of street railway property will decrease as traffic is deflected.

Before it becomes necessary to provide rapid transit on other levels than the surface, frequently street-car service may be accelerated. In many cities streets are being widened from 50 to 60 ft. to 100 ft. At the time a street is widened, it does not cost much more to make the width 120 or 150 ft. In streets more than 100 ft. wide cars can be operated at much higher speeds notwithstanding the crossing of other streets at grade.

A 100-ft. highway, paved the full width, is dangerous for people to cross; but if it is made wider and is divided into three parts, a roadway for traffic in each direction and a reservation in the center for street-car tracks, the danger is largely eliminated. On the other hand, where the track reservations are in streets only 100 ft. wide the side roadways—less than 30 ft.—are so narrow that other users, particularly the Fire Department, object.

The motor bus will be useful as a feeder and distributor of traffic for rapid transit lines. This combined service can be furnished in some instances at greater speed than by street cars, and unless street railway fares are kept low, the cost of rapid transit, motor-bus service will not exceed street railway service in the larger cities and, of course, will be more satisfactory in congested districts.

WINTERS HAYDOCK,* M. AM. SOC. C. E.—There is a very direct relationship between the rapidly increasing importance of bus transportation on highways and the design of those highways as traffic carriers. The large size of the bus and the high speed which it must maintain in order to fulfill its proper mission bring into prominence this relationship between inter-city bus transportation and the capacity and efficiency of the highways which the buses must use.

It is probably true that the most urgent duty of State highway departments at the present time is to obtain the greatest possible mileage of well-paved highways for the money available. However, it is obvious to any one who observes the present trend of traffic conditions on important trunk highways, and even on some of less importance, that within a short time the most important work of the engineer will be the improvement of these highways as to their traffic-carrying efficiency.

A fundamental principle of highway design, to which sufficient attention perhaps has not been given, is that no highway can be an efficient carrier of traffic in large volume if there is provision for only one lane of moving traffic in each direction. This is true because the various vehicles using any highway will necessarily and properly operate at various speeds, and

* Directing Engr., Dept. of City Transit, Pittsburgh, Pa.

if there is only one lane the slowest vehicle will set the pace or compel the faster vehicles to pass it only through the dangerous expedient of dodging into the lane of traffic moving in the opposite direction. If, however, there are two or more lanes for traffic in each direction, all vehicles may be required to keep to the right-hand lane except when passing slower vehicles. Thus, each vehicle may travel at the speed which is individually appropriate for it and the flow of traffic is made flexible and fluid through the constant ability of vehicles to weave in and out from one lane to another without ever trespassing on the part of the roadway assigned to traffic in the opposite direction.

Evidently to insure two unobstructed lanes for moving traffic on a heavily loaded highway there must be provision for stationary vehicles off the traveled section. It is not possible to prevent the stopping of vehicles. Property owners also have rights to the use of their frontage, and this often means that space along this frontage must be available for standing vehicles. Therefore, the minimum requirement for efficiency as a traffic carrier is that a highway should have not less than two lanes for unobstructed moving traffic in each direction, with provision for standing vehicles on each side and pedestrian ways.

This principle of highway design for traffic-carrying efficiency is so simple and obvious that it would scarcely seem necessary to dwell on it were it not that its importance is insufficiently realized. Highway engineers are giving only scant attention to the problem of securing sufficient right of way for trunk-line highways to permit their proper amplification at such time in the future as it becomes financially possible. They are even building long stretches of new highway on newly acquired rights of way which are too narrow for this future amplification, although they pass through open country where the increased width could be obtained at only a moderate additional expense. Along these new highways, and also along the now open parts of the old highways, new improvements and new villages will be built, which will make the future acquisition of right of way exceedingly expensive. The sooner it is realized by highway engineers that the biggest problem they will have to face in the future will be the amplification of present highways, and the sooner they undertake intensive study of the means of meeting this problem, the less will be the inevitably enormous expense involved.

JOHN W. REID,* M. AM. SOC. C. E.—The City of Detroit is one of the very few of the major cities which own and operate street railway systems serving their entire cities. The Detroit city charter, adopted in June, 1918, authorized the ownership and operation of a street railway system and the creation of a commission for that purpose.

In 1920 and 1921, the Department of Street Railways built a system consisting of 61 miles of track and 30 miles of trackage, and operated it privately under a "day to day" permit. In accordance with the terms and conditions of this permit, the Commission has taken over this system and has built some new trackage in territories annexed to the city subsequent to the city's pur-

* Commr., Dept. of Public Works, Detroit, Mich.

chase in 1922 of the properties, power plant, car houses, yards, and 273 miles of track of the privately owned street railway company. The total trackage is 400 miles with 1 590 cars operating. The fare is 6 cents with a 1-cent transfer charge.

Supplementing this service is a privately and very largely locally owned bus company, operating eight routes, with a total of 350 buses, of which 145 are of a double, and 205 of a single-deck type. The company operates on a day to day permit from the city. A 10-cent fare is charged. Many of the company's lines parallel important street railway lines. This condition has brought rather sharply to the fore the question, "Since the City is in the transportation business, should it not control and operate all types and thus eliminate competition?"

As a partial answer to this question, and also as a practical means of meeting demands for some degree of transportation in newly developed and rapidly developing areas, the Street Railway Department has established a number of bus lines. Several lines have been established in streets having street railway service. These are operated in an effort to furnish both express and local service and to eliminate the "jitney." The buses are well patronized, but the "jitney" persists and in increased numbers. The question of the city's right to effect its order of more than a year ago, banishing the "jitney" from its streets, is now before the Supreme Court of the United States. A decision is looked for during 1927.

The matter of adequate highways has been very carefully worked out in connection with the Rapid Transit Commission's studies for a complete rapid transportation system. The Commission's plans contemplate subways within the 6-mile circle from the center of the city. Outside this area a system of super-highways, having a width of 204 ft. is projected, the center 84 ft. being reserved for 4-track rapid transit on the surface. On each side is 60 ft. for pedestrian and vehicle traffic. The intersecting highways, at approximately ½-mile intervals, will pass under the super-highway, thus allowing uninterrupted movement to high-speed through-traffic vehicles. Woodward Avenue, from the north city limit of Detroit to the City of Pontiac, a distance of approximately 18 miles, has been developed in this fashion during the past three years, except that intersecting highways are not separated.

Super-highways are proposed every 3 miles at right angles to one another. Intermediate 120-ft. highways will be located on each mile line and 86-ft. highways on each half mile, local service streets (so-called) being 60 ft. wide. This general plan has been adopted by the Common Council of the City of Detroit and the Wayne County Board of Supervisors as the plan to work to. At its 1925 session, the Michigan Legislature passed a law authorizing adjoining counties, through their Boards of Supervisors and Road Commissions, to join for the purpose of securing rights of way by dedication, purchase, or condemnation. The Counties of Wayne, Oakland and Macomb have, under this authority, acquired a very considerable mileage of such right of way by dedications.

In connection with its subway route studies, within the Six Mile Circle, the Rapid Transit Commission also developed an interior thoroughfare plan, so-called, which is a very valuable contribution to the solution of Detroit's street problem in the older and densely settled areas of the city. The older part of the city is laid out over old ribbon-like French farms, which extended back from the Detroit River from 1 to 3 miles. Uncontrolled platting of these by the original and successive owners accounts in large measure for many of the street openings, widenings, and straightenings necessary to afford a reasonably good arterial system for both transportation and traffic.

The problem of the parking of automobiles, particularly that falling in the "storage" classification, in the streets of the city, is a most serious one. It is a matter of tremendous import to both trade and traffic. The city is spending approximately $3 000 000 annually in widening existing street pavements to provide greater freedom and safety in traffic movements, but the new pavement areas thus obtained are quickly seized upon as "storage" space for automobiles. Notwithstanding this, the traffic capacity has been materially increased; but the "parking" problem is yet to be solved in Detroit, the center of the automobile industry.

LUCIUS S. STORRS,* ESQ. (by letter).†—The status of the automotive vehicle in public transportation service has been thoroughly and convincingly presented by Mr. Budd. Its true economic position will not be determined until a full realization is had of the fundamentals expressed in his conclusions.‡

The rapid expansion of American cities has been primarily because public transportation is available to every resident of the community. Maximum convenience and efficiency of such public transportation can be accomplished only when all such agencies are under the direct control of one able transportation group whereby the maximum efficiency of each unit can be planned with direct relation to all other such units.

The automotive vehicle presents an exceedingly valuable aid in expanding public transportation throughout an entire urban area. It performs a higher grade of service than is rendered by rail units—a kind that will attract a large portion of the public accustomed to using private means of transportation. In this way the motor bus or motor coach will serve the communities themselves by making possible a reduction in the number of private automobiles utilizing the public ways.

---

* Managing Director, Am. Elec. Ry. Assoc., New York, N. Y.

† Received by the Secretary, June 16, 1927.

‡ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 818.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE LOGGING AND LUMBERING INTERESTS OF THE PACIFIC NORTHWEST

A SYMPOSIUM

Discussion*

By Messrs. Roland H. Stock, Hugo Winkenwerder, and C. S. Chapman.

Roland H. Stock,† M. Am. Soc. C. E.—The great variety of physical conditions and of timber in the Northwest make logging operations considerably different from those elsewhere, so that the work of the engineer is extremely varied. In the old days most of the logging was done at tide-water and along the banks of navigable streams. The equipment was simple and the engineering was even simpler. About the only engineering structures built were log dams from 10 to 50 ft. in height, used to store water which, when released, assisted in floating logs to market. Now, the timber stands are away from streams mostly in the extremely rugged territory of the Cascades and Olympics, so that the larger operators have come to see the necessity of more thorough engineering advice.

Formerly, in starting a new district, it was customary to locate railroads on a 5% grade, to keep the curvature as low as possible, and to proceed as far as convenient. Even a few years ago, the logging superintendent would decide where the line was to go, and then hire a surveyor to help lay out the location. Soon they would be halted by a box canyon and have to start a switchback. In one instance when they got to the top of the divide, they found they were too high, so they had to put in a grade adverse to the loads. Latterly, the advice of competent engineers has demonstrated the possibility

* This discussion (of the Symposium on The Logging and Lumbering Interests of the Pacific Northwest, presented at the Summer Meeting, Seattle, Wash., July 14, 1926, and published in May, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Civ. and Hydr. Engr., Aberdeen, Wash.

of attacking this particular problem in an entirely different way, which would have saved its cost in construction on the first mile of road. Furthermore, the correct location would have made available 200 000 000 ft. of timber that cannot be reached with the present location.

Progress, particularly in the line of new machinery, has reached a point where radical improvement is not likely. The field of the engineer lies more in the economical use of that machinery in connection with logging railroads. To the best of the speaker's knowledge little progress has been made to this end. As an illustration of the economies possible, one logging engineer was able to raise the efficiency from 1 200 to 1 800 ft. per man per day. This, however, required six months' work on the management before he could convince it of the feasibility of the plan.

The first requisite in the development of a tract is a complete and reasonably accurate estimate of the timber on the area, both as to types and amounts. Then comes a proper topographical survey, reasonably accurate, depending on the roughness of the topography.

In some of the rough Northwest country there is a field for the air map showing a mosaic of a tract. Since logging is conducted largely by non-technical men, a relief map of the entire tract also has a distinct value in evolving a plan of logging, as it gives a visual representation of the tract itself to a numerical scale.

Flumes are valuable for transportation in the so-called "short-log" country east of the Cascades, but have no value in the "big-log" country. However, as more second growth is logged this method may become of value.

The speaker agrees with Mr. Donovan that reforestation is largely a matter of controlling fires.* At least 75% of the logged-off lands will reforest themselves if given a chance. The first growth is generally hemlock, and, later, fir. With the heavy rainfall, the moderate climate, and a topography unfit for agriculture, the Pacific Northwest should always be the lumber center of the United States.

HUGO WINKENWERDER,† ESQ.—The papers comprising this Symposium are splendid. The various subjects discussed are, however, so specialized that it would be impossible for any one person to discuss all of them intelligently. The speaker, therefore, will confine his remarks to the papers of Mr. Allen‡ and Mr. Woods.§

These papers give much food for thought. It is true, as Mr. Allen stated, that people generally are not familiar with what is going on along the lines of reforestation. It is of course to be expected that the persons engaged in the lumber industry, the professional foresters, and the public, will view this problem from different angles. There is, however, a growing tendency for these groups to understand the other's viewpoint, and they are well on the road toward a common understanding. The more progressive lumbermen have come a long way toward the forester's viewpoint on reforestation as a

* *Proceedings*, Am. Soc. C. E., May 1927, Papers and Discussions, p. 820.
† Dean, Coll. of Forestry, Univ. of Washington, Seattle, Wash.
‡ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 886.
§ *Loc. cit.*, p. 891.

business. While it was difficult for the forester to give up what he considered his idea and "come down to earth", every forester recognizes that the first principle of his profession is that forestry is a business. It is true that many foresters have been more or less in the clouds, but it must be recognized that they, too, have come a long way toward understanding the more practical business viewpoint of the lumbermen.

With reference to the general public, sentiment toward the practice of forestry was never better than at present. This is evidenced by the provisions of the Clark-McNary Act, by the interest displayed by the public in forest fire prevention, and by the reforestation legislation passed in recent years by the various States. These various groups, therefore, are fast approaching a common ground. The difficulty in the way of more thorough co-operation in the past has been a lack of understanding of each other's problems. What the public needs is a little more good sound business education on this problem—something to show how vital the problem is to the public generally, for it is certainly as vital to them as it is to the forester and the lumberman. Just a little better understanding by each group of the other one's special problems should result in the best spirit of co-operation.

Mr. Woods has shown, in his paper, the rapid strides being made by the Long-Bell Company in the extension of its policy of reforestation. This is most illuminating, particularly at a time when so few lumbermen on the Pacific Coast have taken any definite steps toward putting their logged-off lands under permanent forest production.

He referred to the problem of forest taxation. The present system of taxing growing forests amounts to practically the same thing as taxing the farmer on his wheat crop once a month, or even every week, until it is harvested. The young growing crop of timber is not taxed according to its present worth. The general property tax, as applied to growing forests, is fundamentally wrong in principle, and it is this system of forest taxation that is doing more to hinder the owners of timberland from adopting a policy of continuous forest production than anything else. The Federal Government is now making a special study of this problem. Certain States have enacted rather satisfactory laws based on the principle of the yield tax; outstanding examples are the States of Michigan and Louisiana. Many other States are studying the problem, and in the State of Washington considerable progress is being made.

In Washington the problem is one of extreme importance. Timber has, from the beginning, been the most important natural resource. The early industrial development of the State has hinged more largely on its magnificent virgin forests than on all other resources combined. People were attracted in the early days because the forests offered most excellent opportunities for labor, for the investment of capital, and for the development of industry. Even to this day, 65% of the industrial payroll of the State comes from the lumber industry. It is often argued that even if the timber in time will be exhausted other industries are taking the place of the lumber industry so rapidly that

its failure will not materially affect industry. Even such an industry as transportation is largely dependent, in the State of Washington, on forest products; 65% of the freight shipped out of the State, over the four large transcontinental railroads, and 80% of the boat tonnage of Puget Sound are composed of forest products. This is only one illustration of the far-reaching influences of the timber resources in their relation to all industries. Again, probably only 50% of the land area of the State of Washington is suitable for agricultural development, 15% is absolutely waste land, and 35% will be valuable only if kept under timber production. This shows how intimately the forestry problem is involved with the entire land problem.

Although Washington has been used as an illustration, the same principles may be applied to every section of the country where timber is the chief resource. The real solution to the problem is to be sought in a further education of the public, which will lead to proper legislation. The Washington State Legislature has passed an amendment to the Constitution which will pave the way for proper reforestation legislation, but before it can become a law it must be approved by popular vote. This may be done at the general election in November.*

The people of the Pacific Coast region, however, are beginning to realize the necessity for action, and it may be expected that such legislation as is needed will soon be on the statute books. The problem is too big for the State and Federal Government to handle alone. It is generally believed that it will be solved only if each of the three large groups of timberland owners, namely, the Federal Government, the State, and the private timberland owner, all do their share.

Timber growth becomes spontaneous (that is, the forest is naturally regenerated) provided the fire situation is handled properly. Much depends on this. There is a diversity of opinion as to whether or not the slashings should be burned after logging. The burning is a matter of future protection against fires. If the slashings are burned during the first year after logging, quite satisfactory reforestation can be expected, and the fire hazard to the new growth has been eliminated.

Douglas fir seed retains its vitality for seven years. There is quite an accumulation in the ground when the old forest is cut off, and if the fire did not destroy it completely there will be spontaneous growth. Mr. Allen has mentioned the subject of wind-blown seed. This method of regeneration is slower than, but as effective as, natural reforestation, and is very successful.

The utilization of the slashing material is another matter that has great possibilities. It is one of the great problems in forest conservation. Logging methods will also change in the future, and a great deal more of the small material will be utilized. The wood material will also be used in somewhat different form. Many pulp and paper companies use slashings and small-sized trees, and there are extensive possibilities for their further use.

Research work along these lines of utilization of timber waste is most urgent. More is known on this subject concerning Eastern than West-

* The Constitutional Amendment was defeated at the general election in November, 1926. Similar legislation was also defeated in Oregon, but was successfully passed in California.

ern species. Products are manufactured that will produce from 50 to 400% more value than raw lumber. Through extended research, factories could eventually be established to utilize this large amount of waste and manufacture it into products that would be of great commercial value.

C. S. CHAPMAN,* ESQ. (by letter).†—Mr. Woods very properly places strong emphasis on the need for tax reform and better fire protection.‡ These are to-day the two great stumbling blocks to forest practice in the Northwest. It must be recognized that a crop requiring a century to mature should receive every consideration from a taxation standpoint. Especially is this so since the per unit yearly increment of a forest is a very small amount and the land used for forest growing is largely of a sort that cannot be utilized for any other purpose. Instead, however, of encouraging reforestation through enactment of tax legislation to meet a peculiar situation, forest crops are singled out for taxation and the timber is taxed over and over again, whereas most annual crops escape any taxation whatever. Most States are attempting, in one way or another, to remedy this situation, and it is hoped the day is not far off when the forest land owner can definitely compute his annual tax charges, paying on his crop (when cut) a proper proportion of its then value. The public cannot lose by such an equitable arrangement, which will go far toward making forest growing an attractive enterprise.

In the matter of fire prevention, Mr. Woods refers to fires in cut-over and burned-over areas where re-stocking is taking place. Losses of this kind, while at present not computable, or at least practically so, are far too general, and more co-operative effort is required to reduce the damage to a minimum. There is often, however, a mistaken idea as to the loss of mature timber through fires in the Pacific Northwest. In the past ten years, or since patrol and protective effort has become well organized, losses of this nature have been small compared with the value of the resource. In Western Washington, for example, since 1916 the computable monetary loss to logging equipment and logs has been more than three times that of standing timber.

In reality, merchantable timber should not be regarded as a resource seriously threatened by fire as long as efficient protection systems are in effect. The need for public co-operation and increasing expenditure by all agencies to keep abreast of the fire hazard is, however, apparent. Fires in the Douglas fir belt are not of a character that in any way interferes with tourist travel. In fact, fires are seldom seen along the main traveled highways.

The Northwest is quite proud of the record established by its protection organizations. It has gone a long way, although far from all the way, toward the solution of its fire problem, and it is going to pursue the forest taxation question until some rational solution is put into effect.

* Forester, Weyerhaeuser Timber Co., Tacoma, Wash.
† Received by the Secretary, June 13, 1927.
‡ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 891.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOME PHASES OF IRRIGATION FINANCING

Discussion*

BY MESSRS. M. C. HINDERLIDER, C. E. GRUNSKY, JOSEPH JACOBS, AND H. F. DUNHAM

M. C. HINDERLIDER,† M. AM. SOC. C. E.—This paper is an excellent presentation and analysis of this intricate and all-important subject. Doubtless the greatest problem of America to-day is that of the farmer.

While the tribulations of the agriculturist in the humid region have been such as to bring about the abandonment of hundreds of thousands of acres of what formerly had been either fairly profitable investments, or means of complete support to himself and family, the plight in which the farmer in the arid West has been placed for several years past is even worse, due to the relatively greater cost of production, more unfavorable location with respect to markets, and less favorable facilities for financing his needs. It is true that these handicaps have been compensated to some extent by greater crop returns per acre from the more productive soils, coupled with cheaper production of live stock through the use of the great ranges of the West; but such marginal differences in favor of the farmer in the arid region are of minor importance, particularly as regards settlers on partly developed lands not under stabilized production.

In most of the conclusions‡ reached by the author, the speaker is in accord. The first of these is, that "increase of irrigated area will and should be slow, when and where crop prices do not render farming profitable". This is largely true as regards large developments financed by private enterprise; but as to developments of lesser magnitude, the conclusiveness of this statement is largely dependent on what constitutes profitable farming. If this consists

* This discussion (of the paper by D. C. Henny, M. Am. Soc. C. E., presented at the meeting of the Irrigation Division, Seattle, Wash., July 15, 1926, and published in May, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† State Engr. of Colorado, Denver, Colo.

‡ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 912.

entirely in the accumulation of money, then the author's conclusion is logical; if, however, the reclamation of useless lands as an abode for the rearing of a family in any particular community and under climatic and social conditions which in his judgment are most conducive to their welfare, is the principal incentive, then the continued reclamation of lands in any particular community without regard to how profitable such reclamation may be from the standpoint of monetary returns, is justified.

In many localities in which extensive reclamation has taken place through corporate and Government effort, the climatic and social conditions do not appeal to many a capable farmer who would gladly purchase a farm under a newly constructed project in a community in which he has other interests or prefers to remain. Such reclamation development is an important factor in the gradual building up of isolated districts into homogeneous communities which make for stability in commercial and transportation lines and in the diversified pursuits needful for the well-being of a prosperous and contented people.

The speaker is unable to agree wholly with Mr. Henny's conclusion to the effect that "the irrigation district method is not well adapted to the reclamation of large areas of desert land". Many of the largest and most successful developments in Colorado have been effected under the original Irrigation District Act and amendments thereto. These projects embrace a great many thousand acres of some of the choicest lands in the State, and numerous large storage reservoirs, with capacities running into hundreds of thousands of acre-feet. As illustrative of the success of this method of financing and administering irrigation development, one single individual through such means was instrumental, to the time of his death a few years ago, in having placed under irrigation almost as great an area of land as had been served to that time by the U. S. Reclamation projects.

As a means of administration during the colonization and development period, the irrigation district system is generally regarded by irrigationists as the most practicable method. Many large Colorado projects embodied in irrigation districts have either wholly or almost completely retired their outstanding obligations, are administering their affairs in an efficient and economical manner, and have established a credit which enables them to borrow money, when the occasion arises, on a parity with municipalities. It is true that as experience indicates, it will be necessary to amend the irrigation district acts as now adopted in most of the Western States to meet more effectively the problems of financing and administration as necessity seems to require.

The speaker is heartily in accord with the author's view, that "stagnation of irrigation development and the consequent slowing down of Western growth may prove of serious detriment to the Nation", and this being true it naturally follows that Mr. Henny's succeeding conclusion is sound, namely, that "State and National subsidy to irrigation is justifiable".

Recently, much has been heard in regard to the over-development of irrigated regions, especially with reference to Federal reclamation and the Government's attitude toward future reclamation in the West. It seems that

in all these discussions the major purpose and objective of Western land reclamation has been insufficiently stressed. All those familiar with the progress of irrigation development recognize that it has not been accomplished without financial losses and a wastage of no little effort, which, however, has been relatively no greater than the mortality to railways, banking institutions, and mining and manufacturing industries. Yet no sane person would favor for a moment curtailment of development among these other activities.

Whether the reclamation of the arid lands of the Western States, in adding hundreds of thousands of acres of virgin soil, practically inexhaustible in its fertility, will result in a greater financial return on the necessary investment; or whether it were better to attempt to bring back to a proper degree of productiveness the worn out and abandoned lands of the Eastern States, as has recently been suggested; or, again, whether such attempts in either case would result in some financial loss to the funds appropriated for such purpose, is aside from the question.

The question is, whether or not it is needful to the country's welfare, both internally and externally, for this nation to develop along consistent lines which will eliminate radical demarcations between sections of the country, and, if so, whether such necessity may be determined by the same reasonings and units of measure commonly applied to business undertakings.

Next to the air he breathes, man's greatest necessities are food and drink, and his stomach has ever been his most relentless taskmaster. The greatest problem with which man has had to contend, has been that of food supplies. This shadow from time immemorial has laid as a pall on all nations, except possibly America. No other people has escaped the ravages of famine, that dread scourge of nations. With the passing of time, is America to be less fortunate? Future generations only can answer this question.

For two generations this Nation has prided itself on the theory that it is the melting pot in which the peoples of the earth would be blended into a superior race of individuals. It has recently awakened to the fact that in this melting pot an over-burden of slag and an excess of dregs are being accumulated. To-day, the population of many great cities largely consists of peoples who have little or no conception of the ideals on which the Government is founded, with the result that these spots have become the breeding ground for all the destructive elements of society and free government.

Was it not Marcus Cato who, fifty years before Christ, said,

"It is from the tillers of the soil that spring the best citizens and the stanchest soldiers; theirs are the enduring rewards which are most grateful and least envied. Such as devote themselves to that pursuit are least of all men given to evil counsels".

These words are as much a truism to-day as in the time of that old Roman philosopher.

There are those whose conception of National wealth is measured in dollars and cents. They demand that the Nation shall cease spending its own money in turning desert areas into oases until the Government shall have recovered the money expended in the work already accomplished, forgetting that the Nation's wealth and strength are not in its accumulated hordes of gold and

credits, but rather in a united and contented people; that its safety is in their ability to look ahead with that freedom of hope born of greater opportunities made possible through the vision of their leaders.

Federal or State reclamation should not be in the nature of a gratuity. The speaker does believe, however, that the Government is justified in a continuing policy of reclamation development of all worthy projects, the cost of which is beyond the ability of the State or corporations to finance, and that the difference between the legitimate cost of such development and the price at which the farmer can pay for the lands so developed should be absorbed by the Nation at large. There is no valid reason why the Government should cease to provide opportunity for the building of homes in these desert wastes, when practically every other industry of the country is subsidized through high protective tariffs.

Very properly Mr. Henny has stated that any plan for adjusting economic conditions through price fixing or other artificial means is unsound and will eventually bring about its attendant string of evils. That inexorable economic law of supply and demand is no respecter of persons or pursuits, neither can it be evaded without penalty; the law of the survival of the fittest applies to the farmer the same as to any other individual or business. If the individual farmer has it in his make-up to succeed, success will be his; if not, very likely he will fail. Much may be said in favor of the suggested plan for financing practically all the needs of the farmer under the Federal reclamation projects. On the other hand it is urged that such plan is leading directly to a condition fraught with more danger to the country than the incidental loss of a few million dollars invested in reclamation development could be. This refers to the destruction of that self-reliance and freedom of action which has been a characteristic pre-eminent in the builders of the Nation, and especially of the West, and the substitution therefor of a system of paternalism with all its attendant evils.

It is a most needful and sound policy for the Federal Government to build these necessary works, but when this has been done, let them be turned over to the management of irrigation districts, sponsored by the States, and free from Congressional influence, and at a price the land owner can pay. Charge him a graduated scale of payments and a low rate of interest on deferred payments, and require him to meet such payments just as the farmers in the East are required to meet their notes when due.

In a message to Congress, President Coolidge has stated that,

"Efficiency of Federal operations is impaired as their scope of operations is unduly enlarged. Efficiency of State Governments is impaired as they relinquish and turn over to the Federal Government responsibilities which are rightfully theirs."

Farming in the arid regions is one of the most highly specialized industries. Success requires a rare combination of attributes, the chief of which are an inherent love for agricultural pursuits, industry, and thrift. Without these, no farmer can be a success; with these, he will succeed no matter what his environment. The degree of his success, of course, will largely depend on his other qualifications and limitations. Too many people try their hand at

farming after having made a failure at other pursuits; very naturally they fail at farming too. Many then run for Congress.

With a more equitable adjustment of the discrepancy between the cost of production and the gross return for his products, the farmer who is capable will work out his own salvation, with a consequent increase in his purchasing power, followed by a rapid demand for the few hundred thousand acres of irrigated lands now unclaimed. This, in return, will provide the opportunity to finance legitimate reclamation with certainty of a proper return on such investments.

C. E. GRUNSKY,* PAST-PRESIDENT, AM. SOC. C. E.—The Engineering Profession is to be congratulated on the tendency of its members to pay more and more attention to the economic aspects of projects. This is well brought out in this paper.† It takes a long time to bring most irrigation projects to the point at which income will fully cover all expenses of operation. Some good projects in California have taken thirty years to reach this point.

State aid to any project is nothing less than a subsidy or bonus. When the United States entered upon its reclamation policy, and permitted deferred payments without interest, even on the assumption of payments made as contemplated, the bonus to the land owner was equivalent to about one-half the cost of the project.

The question arises, who benefits by the subsidy? The point to be made is that the subsidy given the land owner, as in the case of every such subsidy to farmers, as Mr. Henny has well stated, gives only temporary aid. The bonus cannot be passed on to the land owner's successor. Give the farmer assurance that his output will find a market at good prices, and up goes the value of the land; and the next owner gets no benefit from the improved market. He will soon be confronted by the same difficulties that disturbed the original owner before he obtained a subsidy.

It appears fundamental that, if a subsidy is granted, be it by State or by the Federal Government, it should be for the purpose of extending the area that is available for cultivation—extending the opportunity for home building. This is certainly a worthy object. However, when a subsidy is granted, as Mr. Henny shows, it should be granted with a full consciousness of what it amounts to and on assurance that it is worth while.

JOSEPH JACOBS,‡ M. AM. SOC. C. E.—The author apparently favors Governmental subsidy as a probable necessity. He has indicated the basis on which such subsidy might be granted. The matter of subsidy, however, should be hedged about with the greatest possible care, else its privileges will surely be abused. If a subsidy be granted at all, it should be within clearly defined limits and founded on absolute economic need.

There is an easily discernible and growing public sentiment against subsidies, both inside and outside legislative halls, as a result of which it will be increasingly difficult to secure them for irrigation development. Apparently,

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

† *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 896.

‡ Cons. Engr. (Jacobs and Ober), Seattle, Wash.

there is a definite future prospect of the Federal Government demanding, in so far as Federal reclamation is concerned, that all reclamation projects hereafter constructed repay in full to the Government both the principal and the interest on the construction cost. The speaker further believes that on a long-time payment basis, say, 40 years, and on an interest rate basis that the Government can afford to grant, say, 3½ to 4%, most of the projects that can be justified at all, can work themselves out to economic safety without the aid of any subsidy at all. Such a policy would incline the Government more favorably toward land reclamation and would probably result in a more rational and more rapid irrigation development than would otherwise be possible.

A legitimate exception to this is the desirability that the Government absorb the interest charges attaching to the unsettled lands. This would be justified on the grounds that if settlement be long delayed such accumulating interest charges would become an unbearable burden on the land and the settler; and the Government could afford to pay this relatively small subsidy for the direct economic and military benefits it derives from such agricultural development. In the guaranteeing of such a subsidy the Federal and State Governments might well co-operate.

H. F. DUNHAM,* M. AM. SOC. C. E. (by letter).†—This excellent paper will long be noted for its thoroughness and sound views.

An individual or a corporation investing many millions usually conducts his first studies or investigations rather quietly and without special proclamation of future activity and success. It is true the Federal Government is under certain obligations to its citizens, yet it has a Secret Service of reasonable proportions and thus an opportunity to accomplish many things quietly.

A few years ago the country was startled by reports that disabled veterans were to be taken to the vicinity of Lake Okeechobee by the Federal Government and there cared for in a wonderful climate and given the best of opportunities to recover. Experienced engineers from Western projects were sent to make reports and numerous encouraging accounts appeared in the press. In no way was this district suited for the purpose, least of all from a patriotic viewpoint. Lake Okeechobee and its worst surroundings occupy less than one-tenth of the area of the State of Florida. Practically all the remaining nine-tenths could be made available for agriculture at less cost per acre than would be required to secure equally good financial results in the Okeechobee District.

Recently a great boom came upon Florida and many people were led into extravagance and at least temporary losses. Many wrong statements and unsupported conclusions exerted a great influence on people in other States. There has been a general disappointment, in large part temporary, and in some part due to Government methods, which encouraged ideas varying little from those of the early Spanish explorers—ideas of a Paradise without the trouble of making one.

---

* Civ. and Hydr. Engr., New York, N. Y.

† Received by the Secretary, December 18, 1926.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

STATE RECLAMATION IN WASHINGTON

Discussion*

BY MESSRS. JAMES RHEA LUPER, W. G. SWENDSEN, J. C. STEVENS, O. L. WALLER, AND JOSEPH JACOBS.

JAMES RHEA LUPER,† ASSOC. M. AM. SOC. C. E.—The author has shown the definite policy adopted by the State of Washington in extending State aid to reclamation development.

In other words, Washington has been "attending the school of experience", and the experience seems to have covered the full range of State aid policies with the exception of Oregon's guaranty of interest on irrigation and drainage district bonds. Washington evidently has passed through the experimental stage and the experiment has been expensive. Now the danger is, that instead of profiting by the experience, the people may reverse the policy and drop all development work of this nature for this generation.

It is true that all the low-cost, easily constructed, projects have been built. The development of the remaining projects will present difficulties that require the most thorough planning by experienced men if they are to be successful.

The author asks, "Is there any means other than State or Federal subsidy of building the remaining large projects?"

It seems obvious that there is not a prospective profit in further irrigation development that will attract private capital. The real profit is to the public in the permanent wealth created and in the increased business in all channels that will inevitably follow after the project has been successfully completed. State and Federal aid can only be justified on this sound theory. Further large developments of this nature must depend on such aid.

The problems in irrigation development are many. Ample water supply, adequate drainage, good markets, good engineering design, sound financing,

* This discussion (of the paper by R. K. Tiffany, M. Am. Soc. C. E., presented at the meeting of the Irrigation Division, Seattle, Wash., July 15, 1926, and published in May, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† State Engr., Salem, Ore.

capable administration, the elimination of land speculation, a sound settlement program and confidence, all are essential to success. When any one of these is lacking to any large extent complete success is impossible.

Mr. Tiffany suggests the advisability of re-establishing the Reclamation Board, a non-political organization. This seems to be essential for under the present form of government it is better for the head of the Reclamation Bureau to have some protection against political pressure, and against a misguided public sentiment that may otherwise force the adoption of an undesirable project, or his resignation.

This Board should be clothed with sufficient power to control the administrative officers of the project in which the State has invested its funds until such time as the State has been reimbursed, or until the operation of the project has been turned over to the settlers.

It must not be comprised of politicians, as the word is generally used, for it must maintain continuity of policy through different political administrations. The Board should have authority to issue State bonds to a limited extent in lieu of the direct tax heretofore imposed. The beneficial effect of the development work cannot be felt for many years. This direct tax will fall on many who will receive no benefit, while the bonds may be paid as, or after, benefits are felt.

In his reference to economic need for a project, the author points out an important matter that seems to have been overlooked by many others in the recent irrigation development in the West. The development of irrigated farms ahead of the economic need not only cannot be justified, but the colonization of such projects will be an uphill struggle, if not impossible.

Conditions will change, however; in fact, they are changing now. Food consumption is overtaking food production and improvement may be felt within a year. In five years a very marked change should be experienced. In the development of large projects many years may elapse before the farms will be fully developed. It is difficult to say that substantial production on such projects will come before there is need of the crops produced.

Irrigation Districts.—Under all forms of development, whether through private financing, Federal reclamation projects, or projects financed by State funds, the organization of the water users under the Irrigation District law seems essential. The irrigation district's code as adopted in Washington, Oregon, California, Idaho, and other Western States in effect provides a means whereby an independent branch of government—a municipality—can be formed. The district has power to levy taxes and assume obligations, which are guaranteed by all the property within its boundaries. The indebtedness represents a general obligation of the district, and is paid by taxes levied either on a basis of irrigated area, or as an *ad valorem* tax.

The law presents an excellent method of collecting funds and provides administrative machinery for the operation and maintenance of constructed projects. It is good for financing to a limited extent, but when the indebtedness reaches a substantial percentage of the total value of the property in the district, a dangerous situation is created, and loss of confidence may be

caused with the resulting loss of credit to all settlers regardless of the value of the individual's property.

Oregon has a system of assessing the costs of furnishing water rights in proportion to the benefits received. Many of the projects include lands that had almost full water rights before the project was created. Such land may be assessed only a nominal part of the construction cost for the additional water right to be supplied. Yet when such a district gets into financial difficulties, the failure on the part of some to pay pyramids the assessments on those able to pay until all land values in the district are entirely destroyed. By providing a means for fixing the exact amount that the water user will be required to pay for a water right his credit will be re-established, and much progress will have been made toward making the district function as it should.

Oregon has an example of financing local improvements which can be well applied in reclamation development. This is, the Bancroft Bonding Act. Under its terms, a city sells its general obligation bonds in an amount necessary to finance the improvement, and is, in turn, secured by a lien on each parcel of land benefited. The lien is for a definite fixed sum, and the land owner benefited knows exactly the amount of his obligation. It can be paid in yearly installments, or more rapidly, if desired.

Why is it not practical for a State to follow the same plan in reclamation developments by the issuance of a State bond secured by the land benefited? If thorough preliminary surveys are made—and in this must be included adequate soil surveys—the possibility of including lands that cannot bear the charge levied against them is eliminated, and the district will be free from the "kick-back" of the general obligation that occurs when the indebtedness becomes too high.

By foreclosure of its liens, the State could take the land and re-sell it just as cities have done with many vacant lots. The property will continue to have a value as long as the lien does not exceed the value of irrigated land with a water right in that vicinity.

American farmers have not yet learned co-operative development. Co-operative marketing has been successful only in certain commodities, but, on the whole, the farmers are trained in independent thought and action, and do not enter rapidly into situations in which their success is dependent on the success of others in the same community.

Mr. Tiffany has quoted from the Washington State Law providing for the certification of Irrigation District bonds, but has refrained from criticizing that method of financing irrigation work.

It is to be noted that the law provides for an examination of the project by the State Director of the Department of Conservation and Development, whose report is filed with the Secretary of State; also, that attached to the report are certificates of various State officers to the effect that the project is feasible, that the bonds have been legally issued, that the water right is good, and that all the soils are of certain character. The certificate on the bond is signed by the Secretary of State and the seal of the State is attached.

This law has two purposes: First, to protect the investing public against the purchase of unsound securities; and, second, to promote development by

making good irrigation securities salable. The State disclaims any legal responsibility for payment of the bond, but when the project defaults in either principal or interest it is difficult to convince a holder of certified bonds that the State is not morally responsible for his loss.

The State should not mar its good financial name by allowing its seal to be placed on any bond that is not essentially sound. It is not morally right, and may result in direct financial loss because of the increased interest rate that may be demanded on other financing within the State.

It is to be hoped that Washington will not go out of the Reclamation business entirely, but will profit by her experiences and amend her laws so that these projects may be re-organized and that further reclamation developments may proceed on a sound basis.

W. G. SWENDSEN,* ASSOC. M. AM. SOC. C. E. (by letter).†—The history of reclamation by irrigation in the State of Washington, so well outlined by Mr. Tiffany, is repeated in Idaho and practically in all the arid States.

Much has been said about the failure of irrigation enterprises. Viewed purely from the standpoint of a return on invested capital, a large percentage of the business might well be classed as failure; if viewed, however, in its broader aspects of developing a new country, producing the necessities of life, and increasing the taxable wealth, quite a different story is revealed.

The financial structure on which an irrigation enterprise must rest is based almost wholly on the efforts of the farmers in the area. Unfortunately, a solution of their problems is not found in any concrete formula, but is abstract in the extreme. In weighing the economic feasibility of most irrigation projects, conclusions have, therefore, necessarily been based largely on expectancy. The equation reflecting farm success is filled with many unknowns, not the least important of which is the psychology of the farmer himself, and his fitness and willingness to engage in the activity at hand. It is not surprising, under these conditions and especially in view of the fact that the art is a new one, that forecasts as to the financial outcome of irrigation projects should be found in error.

It has been a common practice on the part of the promoters in the beginning of these enterprises to absolve the engineer from responsibility in the matter of financial return; but, ultimately, and almost universally, where a financial loss has existed, the responsibility for it has been placed with the engineer. It seems worth while, therefore, in professional interest that the Society should give careful attention to this question. In addition to the engineer's direct personal interest, the problem of how and when the natural resources of land and water in the arid States shall be developed, is of sufficient public, State, and National importance to warrant consideration by the highest engineering and business talent.

It is true, as Mr. Tiffany states, that most of the remaining undeveloped reclamation projects are not, under existing conditions, attractive to private capital and that if they are to be constructed soon, governmental aid of some

* State Engr., Boise, Idaho.
† Received by the Secretary, September 21, 1926.

sort is necessary. Unless it is established with very reasonable certainty that additional agricultural products are necessary to the State or Nation, or to the happiness and well-being of the people, there seems to be no justification in equipping this industry with a public subsidy. The use of State or Government funds at interest rates below those prevailing in other business, or with no interest at all, can be nothing but subsidy.

Where a group of citizens are suffering hardships on farms because of insufficient irrigation water or of unoperating irrigation works, there may be some justification in using public funds for their relief, even if this term, "relief", has come within the last few years to be invoked with almost constant regularity.

Whether wisely or not, the writer has urged a policy in Idaho of directing public assistance and attention only to projects already occupied, where distressed conditions exist, considering new and undeveloped areas only to the end that their resources may be available later and unimpaired for orderly development and use when economic conditions warrant.

The engineering and structural problems in reclamation by irrigation may be definitely understood, analyzed, and solved; settling the land and bringing it to a state of profitable production is the difficult and uncertain problem. If the so-called existing "lame ducks" can be so improved that substantially all irrigated areas present successful farming conditions, much will be accomplished in stimulating the settlement of new areas. The matter of investing public funds in such enterprises must be approached with great care so as to provide financial success to the State or Government and to guard against relieving settlers of individual responsibilities they must assume if good citizenship is to endure.

The writer entertains grave doubts as to the advisability of attempting, especially at public expense or under any form of subsidy, to reclaim by irrigation any unoccupied areas until existing projects are settled, and the "lame ducks" improved, especially in advance of a greater demand than now exists for increased agricultural production.

These are to be regarded as an expression of personal views and not in any sense as reflecting public sentiment in Idaho, where, as in Washington, a difference of opinion exists.

J. C. Stevens,* M. Am. Soc. C. E.—The author believes that no more projects should be built until existing ones are entirely occupied. This policy is entirely wrong. People have particular inclinations and diversity of desires, and they will not all go on any one project. One might as well say that no more houses should be built in Seattle, Wash., until all those in Portland, Ore., are occupied.

That same principle will apply to irrigation projects. There is undoubtedly a demand for irrigation lands locally, that can only be supplied by building the smaller projects.

* Cons. Hydr. Engr. (Stevens and Koon), Portland, Ore.

O. L. WALLER,* M. AM. SOC. C. E.—In administering the irrigation loan, the State of Washington has made some mistakes, but they are not serious. After the World War, some banking and business houses in the State found themselves over-extended and had to charge off some bad accounts, but they did not go out of business. Neither should the State go out of the irrigation business as long as it has lands that may be developed by irrigation. It would seem that a State should be interested in the development of its resources, particularly where the necessary expenditure is beyond the reach of private individuals and of private capital.

The time is not far distant when the United States will need more food to supply a growing population, and not only each State, but the United States, should make a study of all possible land developments that may be made, looking forward to the time when this need will be imperative. It is not important whether agriculture is developed through irrigation or by other methods; but it is to the advantage of any State to look forward to the development of all its natural resources just as much as it is to the advantage of an individual to develop his resources.

As compared to other States, Washington has small acreage now under irrigation. However, it has a large area yet susceptible of such development, and its water supply is abundant. That being the case, it should be the policy of the State to render every assistance possible in developing the lands pending the time when the growing National population will demand additional food.

JOSEPH JACOBS,† M. AM. SOC. C. E. (by letter).‡—Are the financial losses and other difficulties which the State of Washington experienced in its reclamation work, as recited by Mr. Tiffany, attributable to defects in the law; to the method of administration; or to other causes? Are they necessary, ineradicable concomitants of State activity in this type of enterprise; or are the evils correctable by a modification of the laws and by a wiser administration of them such as would result from a policy of ignoring politics as a primary consideration in the selection of the administrators? The writer entertains the latter view and believes that the chief difficulties have been political rather than technical, economic, or fatal deficiencies in the law itself.

No accidental or obscure, mysterious causes were responsible for the financial losses mentioned in the paper. The dangers were easily discernible. Practically all the physical and economic difficulties were known in advance and, for the most part, properly weighed and reported by responsible engineers. It is creditable to engineers that they, more than any others, pointed the safe line of attack and sounded warnings. If, however, it is to be the policy of the appointing power of the State to put in control, over the engineer, men selected from the field of politics, with practically no previous experience, and whose acts and decisions may too often be colored by political considerations—then but little improvement can be expected, no matter how good the laws. Graft and willful dishonesty are not here an issue, because no such delin-

* Vice-Pres., State Coll. of Washington, Pullman, Wash.

† Cons. Engr. (Jacobs and Ober), Seattle, Wash.

‡ Received by the Secretary, June 9, 1927.

quencies are involved, but only the questionable policy of appointing to responsible administrative control of essentially technical or scientific departments of government men who have had no real training for such specialized work.

In this situation the Engineering Profession should make its influence felt more effectively. It should seek to have more clearly defined, by statute or otherwise, the required technical qualifications for heads of departments. It should also seek to overcome the mistaken notion, rather widely entertained, that the engineer is necessarily a narrow technician with little qualification for supreme, administrative control of a department of government, even if that department's interests are chiefly scientific and technical. It is true that the engineer is often only a technician, but it is also true that as high an average of capable, business administrators are found in the Engineering Profession as in other business callings—probably more, and certainly more than in the purely political profession. If it were necessary to admit, which fortunately it is not, that there is no hope of improving the quality of American politics, then the State Government ought to be taken entirely out of reclamation activities.

The State's remaining major projects cannot, on account of their magnitude, be built except as public enterprises. If their development is deemed necessary, the primary determination of policy should relate to whether the State or the Federal Government shall undertake them, or whether these agencies, in co-operation, shall do so. Another alternative is the creation of a large improvement district to issue its own bonds and prosecute the work. Such a district would require special legislation for its creation and successful operation and, to be financially practicable, it would need to include already developed towns and other rich, taxable property to give increased security and marketability to the bonds. Such inclusion would be seriously opposed and this alternative, therefore, can be regarded as only a remote possibility.

That the State's interests in these developments—and this applies to all the irrigation States—is more intimate and immediate than those of the Federal Government cannot be gainsaid; but the Federal Government, too, has a vital interest in Western agricultural development, not only as a means of promoting National prosperity, but also in respect to military factors. Almost every consideration of the problem seems to suggest State and Federal co-operation. Because of State constitutional limitations, as well as for other substantial reasons, it would seem desirable that the actual construction of these major reclamation projects should be in charge of the Federal Government, but it is to be hoped that some satisfactory form of co-operation can be evolved that will provide for the States meeting their just obligation in the premises, particularly as relates to the colonization of the projects.

Under these auspices, and provided the Government grants long terms and low interest rates for repayment of the construction costs, it may be anticipated that these developments will be entirely successful. Under the present Federal Reclamation Act no interest is charged, but for large developments of the future it has been suggested from many sources that such interest should

be charged except possibly during the period of construction and of that normally chargeable to the unsettled lands. Even these partial waivers of interest have been objected to by some as being a subsidy; but if, as a result, new values are created that redound to the advantage of the grantor as well as the grantee of the subsidy, it cannot be regarded as a charity. Purely on the basis of paternalistic charity a subsidy is not warranted, but these are not of that class. Should the Government demand interest on deferred construction cost payments, excepting only the waivers already suggested, the Western States, in the writer's judgment, should neither be surprised, nor should they complain, provided only that the time allowance is long and the interest rate low. On the contrary, they should accept such a dispensation as a sound economic basis for future irrigation development and should modify and shape their State laws to that consummation.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

DESMOND FITZ GERALD, Past-President, Am. Soc. C. E.*

DIED SEPTEMBER 22, 1926.

Desmond FitzGerald was born on May 20, 1846, at Nassau, New Providence, Bahama Islands. His father, an officer in the British Army, in which he attained the rank of Captain, was born at Turlough Park, in the western or northwestern part of Ireland, of Irish gentry. Desmond FitzGerald, in 1901, went to this region, enjoyed a pleasant visit with cousins and other relatives, and was able to trace his family back through several generations. The lands which had been long in the family retained several interesting ancient landmarks.

His maternal grandfather, a native of Scotland, had been Judge of the Court of Admiralty and President of His Majesty's Council at Nassau. His mother, although born in Nassau, came of Colonial stock and through her he was a descendant in the seventh generation from Roger Williams, a prominent figure in New England history, who, when banished from Massachusetts on account of his religious and political opinions, established the City of Providence, R. I., and the First Baptist Church there. He secured to Providence Plantations full power to rule themselves, and was the first Governor of Rhode Island. As late as 1923, Desmond FitzGerald owned several lots of land in Providence, part of a tract which was granted to Roger Williams and had never been out of the family.

Mr. FitzGerald thus had an unusually fine inheritance, and it is not to be wondered at that he was able to maintain an independence of action and to secure a freedom from improper interference with his duties as Engineer or Superintendent by members of committees or commissions whose experience was political rather than technical. His success in this way was so striking that it became the subject of comment.

In 1848, after the death of his grandfather, the family moved to Providence, where they lived for many years.

Mr. FitzGerald's early education was received in the public schools of Providence. He did not receive college training either in the arts or engineering. Following his Providence schooling, he spent a year studying art in France, as at that time he desired to become a sculptor. On his return to the United States he entered Phillips Academy, at Andover, N. H., from which he was graduated in 1864. At this fine Academy he evidently improved his opportunities to learn how to study and acquired the habit of thoroughness. After a short interval, and while still under age, he became Deputy Secretary of the State of Rhode Island, and, in 1866, was Private Secretary to

* Memoir prepared by the following Committee: George F. Swain, Past-President, Am. Soc. C. E., and C. Frank Allen and William E. Foss, Members, Am. Soc. C. E.

General Burnside, then Governor of the State. During this period he found time to engage as "student" in the engineering office of Cushing and Dewitt. Here, again, he was fortunate, or wise, for the Cushing office was for many years the leading engineering office in Providence.

In 1867, he was enterprising enough to start West to seek his fortune, first as Axeman on the Indiana and Vincennes Railroad. It has been stated that the fact that his stakes were better formed and better marked than was usual brought him to the notice of his Chief. A mastery of Henck's Field Book also gained him a desirable reputation, and within a year he became Locating Engineer. He was engaged on various railroads in the Middle West as Locating, Division, or Assistant Chief Engineer, and for a time was in the Federal employ on river improvement work.

In 1871, he returned to Boston, Mass., residing in Brookline where he lived until his death. He soon became Engineer of the Boston and Albany Railroad, accepting this position, which was none too well paid, in preference to a Government position on construction in Boston Harbor; also, in preference to a partnership in an architect's office. Much important construction work was in progress on this railroad, and Mr. FitzGerald, still a young man, was fortunate in having as Consulting Engineer the late Edward S. Philbrick, M. Am. Soc. C. E., upon whom he could rely in part.

His work on this railroad brought him in contact with the late Joseph P. Davis, M. Am. Soc. C. E. This led to his appointment as Superintendent of the Western Division of the Boston Water-Works, which included the Brookline and Chestnut Hill Reservoirs, Lake Cochituate and the connecting aqueduct, embracing substantially all the sources of supply of the city. This position he held from 1873 until 1898.

While in office as Superintendent, he was the active factor in overcoming a reluctance on the part of the City authorities (other than the City Attorney, who supported him) in bringing suit to secure elimination of pollution of the water supply, partly by manufacturing wastes and partly by municipal sewage. This was the case of Pegan Brook in Natick, Mass. Upon Mr. FitzGerald fell the task of securing evidence and tracing actual cases of pollution to their source. The case, perhaps the first of its kind in the United States, after seven years of litigation, resulted in a final decision by the Supreme Court of the State in favor of the City of Boston.

Desmond FitzGerald was the pioneer in the study of color in water and methods of reducing it, resulting in the draining of swampy lands within the water-shed contributing to the water supply, and also to take advantage of the sunlight in bleaching storage waters, as well as stripping top soil and humus from reservoir beds.

He was the first to establish a biological laboratory in connection with water supply and to study especially the bacteriology of water, employing the late George C. Whipple, M. Am. Soc. C. E., then a young graduate in civil engineering, to carry on that work. This was the first and for many years the only biological laboratory operated in this country in connection with a water-works system. He made other studies and investigations in hydraulic

engineering, along lines closely connected with his work. His paper on "Evaporation"* received the Norman Medal in 1887, and has recently been closely confirmed by laboratory experiments. A later paper, "Rainfall, Flow of Streams, and Storage",† also received the Norman Medal, this time in 1893. Other worthy papers along similar lines were presented by him before this and other societies. His studious habit of mind is well attested by these papers and his investigations while in the employ of the City or the later Commission.

When the old system of supply for Boston was absorbed under the newly created Metropolitan Water Board in 1898, its Chief Engineer, the late Frederic P. Stearns, Past-President, Am. Soc. C. E., retained Mr. FitzGerald as Engineer of the Sudbury Department. During his employment with this Board it was understood and provided that he was free to act as Consulting Engineer for other municipalities, and he was actively engaged in this outside work.

During his terms of service on the water supply of Boston he had charge, as Resident Engineer, of the construction of some of the most important storage reservoirs of the System.

In 1902 he resigned from the position which in one form or another he had held since 1873. Previous to this time he had been given leave of absence for six months in 1899 to become Consulting Engineer to the Chicago Drainage Canal, receiving a salary reputed to be the largest paid to any civil engineer up to that time.

He was a member of a Commission under the Secretary of War, to investigate the Lydecker Tunnel, designed by the United States Corps of Engineers, and then under construction for the water supply of the City of Washington, District of Columbia.

The City of San Francisco employed him in connection with its water supply in the important matter known as the Hetch Hetchy case.

He served as Consulting Engineer on the water supply and sewerage of Manila, in the Philippines, involving many questions comparatively new in character. Incidentally he was of considerable assistance in connection with an important highway project there. Later, he was called as one of the experts for the Metropolitan Sewerage Commission of New York with relation to the proper standard of purity for the waters of New York Harbor.

He acted as Consulting Engineer for a considerable number of cities and towns in other States as well as in Massachusetts. He was also retained, and appeared as expert, in a number of noted water cases.

An invitation to become a Lowell Institute Lecturer in Boston is a recognized honor, and Desmond FitzGerald delivered four lectures in a course of twelve on Engineering, his subject being "Water-Works." That his record as a civil engineer was highly distinguished cannot be questioned.

He also set a high standard of public-spirited citizenship. For the Commonwealth of Massachusetts he served for several years as Chairman of the

* *Transactions*, Am. Soc. C. E., Vol. XV (1886), p. 581.
† *Loc. cit.*, Vol. XXVII (1892), p. 253.

Topographical Survey Commission under the direction of which the New York-Massachusetts State line was re-run. This was made difficult by the fact that the important State monuments had been forcibly removed and shifted far from their proper positions. The difficulty was overcome and the line, although at first rejected, was afterward accepted by the State of New York. The re-setting of town bounds also presented problems which were successfully met. He was an active and a valuable member of the Metropolitan Improvement Commission of Massachusetts, in which capacity he not only gave much attention to the railroad problems, but also made a study and a report on foreign harbors.

In the Town of Brookline, where he made his home, Mr. FitzGerald was Chairman of the Park Commission from 1897 until his health made it advisable for him to retire in 1926. He was also President of the Civic Society of Brookline and was one of the Trustees of the Public Library from 1887 to 1926. He was one of the Trustees of the Walnut Hills Cemetery, under town control from its establishment in 1875. He and the two other members first appointed served continuously together for more than fifty years until his death.

When the Town Meeting became so large that a smaller representative body was elected, he was chosen as a member, and had much influence in its meetings. He was unusually effective in securing what he felt was needed in connection with his official interests even when considerable opposition was manifested. During the World War he drilled with the local unit, lectured on patriotism, and was otherwise active for the cause.

Not altogether from self-interest, Desmond FitzGerald was a member of many societies and semi-public bodies, not all of them professional; he gave as well as received.

He was, at the time of his death, third in seniority of service on the Corporation of the Massachusetts Institute of Technology, where he was an active and influential member; a Fellow of the American Academy of Arts and Sciences; Fellow of the Royal Meteorological Society of England; an officer in the New England Meteorological Society; member of the Council of the Society of Colonial Wars, of the Massachusetts Society of Sons of the American Revolution, and of the New England Historical Genealogical Society, to which he contributed in 1911 a booklet showing much research as to his ancestry both in Ireland and the United States. He was also a member, and at one time Councillor (on Art), of the Appalachian Mountain Club, with the interests and activities of which he had a natural sympathy. In 1925, in his eightieth year, he spent the summer months in the mountains of the Pyrenees, traveling partly on foot, partly on mule-back.

Desmond FitzGerald was a lover of art. When eleven years old he went to Paris and spent a year there in school. His cousins with whom he resided were people of wealth and influence, and this, in connection with the opportunities which Paris afforded, may have had great influence on his taste for art and his love of it. He seemed then much interested in sculpture, but finally chose engineering as a career. He could undoubtedly have been an

artist of note had he chosen that field. His circumstances as well as his taste and temperament, later in life, qualified him to become a patron of art as well as a connoisseur and critic, and in these combined capacities he was a leading figure around Boston. Dodge MacKnight, a brilliant water color artist, now most highly regarded in Boston, received his first substantial recognition from Mr. FitzGerald, who not only led in public encouragement of the artist, but also acquired and owned three-score of his pictures, covering various stages of his development; and, in addition, he was the author of a monograph on Dodge MacKnight, privately printed and circulated.

Desmond FitzGerald also was among the first in this country to appreciate the early impressionist, Claude Monet, and others who followed in the same school. He was on terms of personal friendship with MacKnight and Monet, and also with John Singer Sargent.

To accommodate his paintings, he built an art gallery, 27 by 61 ft., adjoining his residence, which later proved all too small for his collection. This gallery was open to the public. He was a Trustee of the Boston Museum of Fine Arts. Following his death, the Trustees adopted the minute of which a part is as follows:

"By the death of Desmond FitzGerald on the twenty-second of September the Museum lost a friend who was deeply interested in its work. In 1916 he was appointed a member of the Board of Trustees by the Institute of Technology, a position which he held up to the time of his death. From 1914 he was a member of the Visiting Committee of the Department of Paintings, and for the last seven years of his life he served as its Chairman. He was affiliated with a great variety of institutions and commissions, and was more or less associated with all the societies that had to do with art in Boston. As a member of the Copley Society he was for years the Treasurer of the committee that had charge of its many memorable loan exhibitions; and he was always generous in lending his own pictures to those exhibitions, and in lending them to other exhibitions where he thought the public would have a chance to see and enjoy them. He had great sympathy with young painters, and he bought their work and did all that he could in other ways to help and encourage them. He was an extensive traveler and traveled intelligently, for he cared for the things he saw from the point of view of an artist as well as from that of an engineer. He loved Nature and even more he loved its pictorial representations, and in his museum in Brookline he had a remarkable collection of modern landscapes and other paintings, and some fine examples of Oriental pottery. He was an optimist enamored of the fine arts, and was kindly, courteous, and cheerful, for to him art was always a joyful inspiration—an inspiration that added greatly to his gaiety and happiness throughout his long and useful life."

Mr. FitzGerald was not without ability as an amateur artist, and his crayon portraits of members of his family were to be seen on his walls. He traveled much, and his artistic taste and his skill as a photographer resulted in several interesting illustrated lectures, mainly if not altogether to his friends, personal and professional.

His professional activities were constant. He was active in the revival of the Boston Society of Civil Engineers in 1873 and 1874, and was among the earliest of the younger men to be elected President. Later, he was elected an Honorary Member. For many years he had made a liberal annual contri-

bution to the Society's permanent fund, and remembered it in his will. He also, in 1910, established a fund for the award of a bronze medal annually to the author of the best paper of the year; a medal of bronze which could be kept in sight, rather than of gold usually placed in a safe for protection, the honor rather than the value being the essential point.

Mr. FitzGerald had been President of the New England Water Works Association, and Honorary Member of the American Water Works Association, and was also a member of the American Institute of Consulting Engineers.

As President, Honorary Member, and twice Norman Medallist of the Society, he attained a unique distinction. His standing as an engineer is secure.

Of clubs, he had membership in the Union, St. Botolph, Boston Art, Engineers', and the Country Club of Brookline. At the time of his death he was Senior Warden of St. Paul's Protestant Episcopal Church, of Brookline.

Slight in stature, he had an abundance of physical and mental energy, and accomplished much in the various lines of his activities, and his genial presence supplemented his other fine qualities. As a public citizen, few gave more for the betterment of the community. In his love for art and its encouragement, and in his church work, he qualified in the finer things of life, artistic and spiritual.

Desmond FitzGerald was married in 1870 to Elizabeth E. C. Salisbury, and is survived by two daughters and two sons.

He was much more than a mere technical engineer. He was a broadly cultivated man with many points of contact outside of engineering. He was thoroughly human, not enclosed in a rigid shell of mathematics or technical detail. Great as he was in engineering, he could have been great in other fields. He was delightful as a companion and had hosts of friends, in the hearts of whom his passing will leave a void which can never be filled.

Mr. FitzGerald was elected a Member of the American Society of Civil Engineers on September 3, 1884, and an Honorary Member on October 15, 1923. He served as Director from 1892 to 1894, as Vice-President from 1895 to 1896, and as President in 1899.

CORNELIUS CONWAY FELTON BENT, M. Am. Soc. C. E.*

DIED NOVEMBER 11, 1926.

Cornelius Conway Felton Bent was born in Boston, Mass., on July 3, 1849. He was the son of George Elbridge and Martha Conway (Felton) Bent, both natives of Massachusetts and descendants of old New England families of English extraction.

Mr. Bent was educated in the Latin School of Boston until his family moved to Philadelphia, Pa., where he entered Chase School, a private institu-

* Memoir prepared by H. A. Lane, M. Am. Soc. C. E.

tion, from which he was graduated at the age of seventeen. Circumstances prevented him from following his ambition of attending Harvard University, of which his uncle, Cornelius Conway Felton, had been President at one time, and, in 1866, he entered the service of the Port Deposit and Perryville Railroad Company, in Maryland, as a Chainman. He continued with this Company until 1867 when he accepted a position with the Chester Creek Railroad Company, in Pennsylvania, having been, successively, Levelman and Transitman with this Company on location and construction work.

In 1869, Mr. Bent went West to become a Transitman on Location, and Engineer in Charge of Construction of the Lake Superior and Mississippi Railroad. He remained in the service of this Company until 1870, when he was appointed Chief Engineer of Location and Construction for the Lancaster Railroad Company, in Massachusetts. In 1875, he entered the service of the Pennsylvania Railroad Company as Assistant Supervisor and was promoted to be Supervisor in 1876. He remained with this Company until 1879, when he became Assistant Engineer for the Columbus, Chicago, and Indiana Central Railway Company, continuing with that Company until June, 1881.

At this point in his railroad career, Mr. Bent transferred his activities to the field of operation, holding, successively, the following positions: From June, 1881, to October, 1882, he served as Superintendent of the Chicago Division, of the Columbus, Chicago and Indiana Central Railway Company, and from October, 1882, to January, 1884, he was Superintendent of the New York and New England Railroad Company. From April, 1884, to October, 1885, he held the position of Superintendent of the Louisville, New Albany and Chicago Railroad Company, and from December, 1885, to 1889, he was Superintendent of the Ohio and Mississippi (now the Baltimore and Ohio) Railroad Company. Mr. Bent was engaged as General Superintendent of the Ohio and Mississippi Railroad Company from 1889 to February, 1894, when he was appointed Superintendent and General Agent of the Baltimore and Ohio Railroad Company at Philadelphia, where he remained until April, 1905.

From April, 1905, to January, 1907, he was General Superintendent of the Baltimore and Ohio Railroad Company at Baltimore, Md. In January, 1907, he was appointed General Manager and transferred to Cincinnati, Ohio, where he remained until July, 1910, when he was made Vice-President of the Staten Island Railway Company and General Superintendent of the Baltimore and Ohio Railroad Company, New York Division, at New York, N. Y. Mr. Bent held this position until January, 1916. In April, 1916, he was appointed General Agent of the Baltimore and Ohio Railroad Company, at Philadelphia, which position he held at the time of his death.

About eight years before his death, Mr. Bent was compelled to undergo a serious surgical operation which weakened him physically for the remainder of his life. His death resulted from pneumonia after an illness of only four days.

He was a man of sterling integrity, well liked by his associates and superiors, and famed for his justice and fairness to his subordinates. That

he was also a man of great ability, is evidenced by his rise from one of the lowest to one of the highest positions in railroad service. He belonged to several New York and Philadelphia clubs.

Mr. Bent was elected a Member of the American Society of Civil Engineers on May 1, 1889.

WILLIAM HENRY FINLEY, M. Am. Soc. C. E.*

DIED MARCH 17, 1926.

William Henry Finley was born near Delaware City, in New Castle County, Del., on January 22, 1862. His father, William Francis Finley, was born in County Galway, Ireland, and his mother, Mary Mortimer McDonough, was born at Castle Bar, County Mayo, Ireland. His parents came to the United States in 1835, and settled in New York, N. Y., but later moved to a farm in Delaware where William was born.

When the boy was 13 years of age the family moved to Wilmington, Del., where he attended High School, finishing at the age of 16. He then secured employment with the *Delaware Gazette* as a printer's "devil". He did not remain long in this capacity, however, for at the end of four years he had become a journeyman printer. By this time he had concluded that the printer's trade was not to his liking.

Mr. Finley had a natural aptitude for mathematics and drawing and while working on the *Delaware Gazette* he began studying drafting at a night school. Subsequently, he entered the employ of the Edge Moor Iron Company as a blueprint boy, doing some tracing in spare time. He continued to study at nights, most of the time without an instructor, a fact that caused his associates to become interested in him and led them to assist him in his efforts to acquire an education.

The Edge Moor Iron Company had at that time one of the largest bridge shops in the United States and many large and notable structures were fabricated by it. Mr. Finley had the opportunity of seeing these structures designed and manufactured. While he was engaged with the Company, it was building the steel structural work for the Brooklyn Bridge, and he had a part in this work.

He remained with this Company until 1887, at which time he accepted a position with the Engineering Department of the Chicago, Milwaukee and St. Paul Railway Company, where his opportunities and responsibilities were greater. He was an indefatigable worker and took full advantage of the opportunities presented to him in this position with the result that, in 1892, although only 30 years of age, he was offered the position of Engineer of Bridges of the Chicago and North Western Railway System, then aggregating about 6 000 miles of lines. He accepted this position and, in 1901, was promoted to be Principal Assistant Engineer of that road. He left the employ of the Chicago

* Memoir prepared by a Joint Committee of the Society and the Western Society of Engineers, consisting of E. T. Howson, *Chairman*, and I. F. Stern, Members, Am. Soc. C. E., G. W. Hand, Assoc. M. Am. Soc. C. E., and Charles F. Loweth, Past-President, Am. Soc. C. E.

and North Western Railway Company in 1905 to become Vice-President of the Widell-Finley Company, a contracting organization. After about a year with this Company he entered consulting engineering work.

However, in 1906, Mr. Finley returned to the Chicago and North Western Railway Company as Assistant Chief Engineer and, in 1913, was promoted to be Chief Engineer, which position he held until shortly after the railways were taken over by the Government in 1918, when he was elected President of the Chicago and North Western Railway Company. In 1922, he was also elected President of the Chicago, St. Paul, Minneapolis and Omaha Railway Company. Because of failing health he resigned from these positions in 1925 with the intention of taking a rest for a year or more. The constant demands for advice and consultation, however, prevented him from realizing his intentions, and he finally opened an office as Consulting Engineer, in Chicago, in November, 1925. He died on March 17, 1926.

Mr. Finley was a great lover of the sea and as a boy spent much of his time about the shipbuilding works in Wilmington, gaining information which was of value to him in later years. As an illustration, when Bridge Engineer of the Chicago and North Western Railway Company he was confronted with the problem of replacing a drawbridge extending across the Chicago River at the entrance to the Passenger Station in that city. It was necessary to develop a method of erection which would not interfere with the operation of the structure it was to replace, while the track arrangement would not permit of it being placed anywhere except on the alignment of the old bridge. Drawing on the recollections of his boyhood days, Mr. Finley conceived the idea of erecting the new structure on ways on the bank of the river some distance from the site that it was to occupy and launching it as he had seen ships launched. This idea, while then novel to bridge builders in the West and regarded with no little skepticism by them, was successful as well as economical.

His later railroad connections were at a time when the railroad companies were rapidly expanding their lines and their engineering activities were extensive. Mr. Finley was connected therefore with a number of large engineering structures, including the Des Moines River Viaduct, at Boone, Iowa; several vertical lift bridges; the controlling of the channel of the Missouri River in the vicinity of several bridge crossings; and other equally important projects. To all of these he contributed largely and efficiently. He was one of the pioneers in the development of the rocker-bearing to take care of expansion in steel structures and his efforts in connection with cylinder piers in bridge foundation work did much to bring this form of construction into more general use.

Although Mr. Finley never had the opportunity to attend a technical school or college, he was always of the opinion that they were of great value and preferred to select his assistants from among the ranks of engineers who had been graduated from college. He possessed and applied a large measure of energy and had great strength of purpose, coupled with an abundance of initiative. Throughout his life he laid great stress on the necessity for the development of initiative and originality. One of his earlier associates writes of him:

"I first met Mr. Finley when I went to Edge Moor as a Draftsman during September, 1886. He had then been in the office for some time and was probably the best Draftsman in the office as far as neat and rapid production was concerned. As he had not had the advantage of a technical education, he was not versed in the methods of stress calculation and design. For some months I observed that he seemed to think that there was little prospect of substantial advancement for him. My roommate and I easily induced him to take up more advanced studies of immediate interest to his work and he spent many evenings in our room, studying and talking about theory and design. I believe that previous to this period he had been studying mathematics alone and with some instruction from others. I remember those evenings with great pleasure, for his personal charm won our affections, and he was unusually quick to understand things then new to him."

A man of broad human sympathies, Mr. Finley was ever interested in the activities of his fellow men. It overflowed the limits of his own professional activities, but did not stand in the way of his giving to the organized efforts of his own chosen profession a very large measure of his time and interest. He was, therefore, active and helpful in the affairs of engineering societies in which he held membership, and his contributions in that respect were notable.

Mr. Finley became a member of the Western Society of Engineers in 1894, was made Second Vice-President in 1900, First Vice-President in 1901, and President in 1902. In addition, he served on a number of its important committees, contributed several notable papers, and frequently discussed constructively papers submitted by other members. His interest in the Western Society of Engineers was abiding; and at the time of his death he had in preparation a paper to be presented to that Society containing the results of investigations he had made personally in Europe with respect to recent developments of motive power for railway purposes.

He was an interested member of the Society, and at the time of his death was serving his second term as its representative on the Washington Award Commission. He was also for many years a member of the American Railway Engineering Association and had contributed to its technical work. He became interested in the American Association of Engineers early in its formation and was its President in 1918 and 1919 when his wise and sympathetic counsel were of great benefit.

On March 8, 1883, Mr. Finley was married to Sarah H. Ferré, at Philadelphia, Pa., who, with four children, three sons and one daughter, survives him.

Mr. Finley was elected a Member of the American Society of Civil Engineers on February 4, 1903.

GEORGE ALEC HARWOOD, M. Am. Soc. C. E.*

DIED NOVEMBER 4, 1926.

George Alec Harwood was born on August 29, 1875, at Waltham, Mass., the son of Alexander Thorndike and Emma Dean (Smith) Harwood. He

* Memoir prepared by George W. Kittredge, M. Am. Soc. C. E.

was graduated from the Waltham Public Schools and the English High School of Boston, Mass., and then entered Tufts College, Medford, Mass., from which he was graduated in 1898 with the degree of Bachelor of Science in Electrical and Civil Engineering. In 1901, he received the degree of Master of Science and in 1913 the degree of Doctor of Science from the same college. During his high school and college career he was able, by his high scholarship, to earn awards that made easier for him the problem of financing his educational adventure.

In 1893 Mr. Harwood entered railroad service as a Chainman on the Fitchburg Railroad, where he continued as Rodman and Assistant Engineer during his college vacations and after his graduation.

In 1900, in response to an advertisement, he became a Draftsman in the Engineering Department of the New York Central and Hudson River Railroad Company. He successively filled positions as Chief Draftsman; Assistant Engineer in charge of construction at and near Syracuse, N. Y.; Assistant Engineer with special assignments from the Chief Engineer; Engineer of Grade Crossing Elimination; Assistant Terminal Engineer; and Terminal Engineer.

In November, 1906, Mr. Harwood was appointed Chief Engineer, Electric Zone Improvements. In 1916, he became Engineering Assistant to the Vice-President of the New York Central Lines and, during Federal Control of the railroads, he served as Engineering Assistant to the Federal Manager of the New York Central Railroad, and as Corporate Chief Engineer of the various companies comprising the New York Central System Lines. In 1920, he became Assistant to the President, New York Central Lines, and in April, 1924, he was elected Vice-President, in charge of improvements and development, which position he held at the time of his death.

Mr. Harwood was a Director of the Monongahela Railway Company; the New York and Fort Lee Railroad Company; the Pittsburgh, Chartiers and Youghiogheny Railway Company; the Raquette Lake Railway Company; the Chartiers Southern Railway Company; the Kanawha and West Virginia Railroad Company; the Michigan Air Line Railroad Company; the Fulton Chain Railway Company; the Hudson River Bridge Company, of Albany, N. Y.; the Hudson River Connecting Railroad Corporation; the Evansville, Mt. Carmel and Northern Railway Company; and the New Jersey Junction Railroad Company. He was also a Director of the Scarsdale Improvement Corporation; the Scarsdale National Bank; and the Mutual Fire, Marine and Inland Insurance Company.

He was a member of the American Railway Engineering Association; the Academy of Political Science; the Sons of the Revolution; the Century Association; the New University Club, of Boston; the Railroad Club; the Scarsdale Town Club; and the Scarsdale Golf Club. He was also a Trustee of Tufts College and a member of the Alpha Tau Omega Fraternity.

Throughout his railroad life Mr. Harwood was noted for his industry, his thoroughness, his integrity, and his honesty. He was unassuming, but, at the same time, a man of force, ability, and mature judgment. As Chief Engineer of Electric Zone Improvements he was prominent in the develop-

ment of the Grand Central Terminal and the area in its vicinity. In connection therewith, he showed his broadmindedness by placing credit where credit was due in a talk given by him during the period of greatest activity, when there were employed approximately 150 men on engineering designs and plans, 100 on supervision, and 3 000 on field construction. He then stated:

"If this terminal is a success, it will be so because it is not a one-man affair, but a splendid piece of team work in which the thought and experience of the operating man, the traffic man, the business man, the lawyer, the architect, and the engineer have been combined".

The following is extracted from the Minutes of the Board of Directors of the New York Central Railroad and the other roads comprising the System Lines, at meetings held subsequent to his death:

"Mr. Harwood ranked high in his profession as a civil engineer. His knowledge of engineering, however, was but the foundation of his great ability in solving difficult problems and of his capacity to bring to successful conclusions important undertakings. He had an all-round and well-balanced mind. His breadth of vision, his grasp of affairs, his sound judgment, his industry made him a unique man. That he was such a man was demonstrated by the work which he did during the reconstruction of the Grand Central Terminal; and this was fully confirmed by his entire subsequent service."

Mr. Harwood's interest in the Village of Scarsdale, in which he lived, was always great. He freely rendered generous service in connection with the financing and installation of the sanitary sewerage system; with securing property for the Athletic Field and the adjoining area for park purposes; with shaping the policy of the village as expressed by the Charter Commission; with educational matters, having served for many terms as President of the Board of Education; with problems relating to village planning; and with improvements in many directions throughout the neighborhood where the results of his wise counsel and advice may be noted. As a citizen and as a property owner, he was active in all things pertaining to its welfare.

In 1900, Mr. Harwood was married to Grace Isabelle Hawley, who, with two sons, Herbert Hawley Harwood and Robert Thorndike Harwood, survives him.

Mr. Harwood was elected an Associate Member of the American Society of Civil Engineers on February 5, 1902, and a Member on February 5, 1907. He served as a Director of the Society from 1915 to 1917.

CHARLES ALBERT JUDSON, M. Am. Soc. C. E.*

DIED SEPTEMBER 30, 1926.

Charles Albert Judson was born on August 11, 1856, on a farm near Florence, Ohio. His early education was obtained in the district schools of Erie County. He lived and worked on the farm until he was twenty years of age, after which he taught school for several winters. He then attended

* Memoir compiled from information on file at the Headquarters of the Society.

the Academy at Delaware, Ohio, for a year, and spent four years in Oberlin College, having been graduated from the latter in 1882, with the degrees of Bachelor of Arts and Master of Arts.

Immediately after his graduation, Mr. Judson took up his residence in Sandusky, Ohio, where he studied and practiced civil engineering. His instructors were Mr. George Morton, and his father, Mr. Albert W. Judson, who at that time, was the Engineer and Surveyor of Erie County. From 1882 to 1884, Mr. Judson acted as Deputy Surveyor, following which he was City Engineer of Sandusky for eight years. In all, he served the City of Sandusky, either as City Engineer or Superintendent of Water-Works, for seventeen continuous years.

In 1902, Mr. Judson was elected State Senator from the Thirtieth District of Ohio. He served in this capacity for two terms, acting on a number of important committees, including those of Geological Survey, Sanitary Laws and Regulations, Railroads and Telegraphs, Roads and Highways, and Drainage and Irrigation. From 1904 to 1913, he was Collector of Customs for the Port of Sandusky.

From the time of his graduation until his death, Mr. Judson carried on almost continuously his engineering work. His surveying and engineering activities covered consulting work for the municipalities of Port Clinton, Fremont, Norwalk, and Huron, Ohio, including plans for, and the construction of, sewers, water-works, and interurban railways.

He made preliminary surveys for what is now the branch of the Pennsylvania Railroad from Sandusky to Columbus, Ohio, and had charge of the engineering on the construction work of that road from Sandusky to Bellevue, Ohio, as well as of the construction of the Lake Shore Electric Railway between Sandusky and Lorain, and between Norwalk and Ceylon Junction, Ohio. In addition, he made preliminary surveys for other lines. His activities also included a variety of bridge work, overhead construction, and street paving. At the time of his death, he was serving as one of the City Commissioners of Sandusky.

Mr. Judson was an active member of the Sandusky Masonic Lodge, and of the First Congregational Church of that city.

He was married to Roxie E. Lowry, of Berlin Heights, Ohio, in 1883. Both he and Mrs. Judson were killed on September 30, 1926, near Belmont, N. Y., when their automobile was struck by a fast freight train of the Erie Railroad Company. They are survived by five children.

One who was an intimate friend and co-worker of Mr. Judson, has prepared the following tribute to him:

"Mr. Judson never laid aside his intellectual interests. He could botanize, work mathematical problems with the children, on occasion read a little Greek and more Latin, and prepare papers for the Literary Club of which he was for so many years a member.

"Mr. Judson lived a successful business life. His services as a civil engineer were valued all through this part of the State.

"He took a man's part in politics. He was Customs Collector and for two terms served in the State Senate. He had the high ideals of political action

that are expected of a scholar, and these ideals rooted down into a steady participation in practical politics year after year.

"He was a strong Churchman, in the choir, in the Sunday School as a teacher, giving unsparing personal attention to all the work of the church. He was an outspoken Christian man, without apology and utterly without ostentation.

"In every sphere in which he worked what he did was characterized by a restrained force and tact that enabled him to see the other man's side of the matter fully and fairly and yet not always share it—if necessary, oppose it vigorously. There are two adjectives that applied to him, 'fearless' and 'friendly'. He was afraid of no one and friendly to every one. He had a keen sense of humor that went with his fearlessness and friendliness. He understood the fine art of getting on with people, and was always ready to take hold with bare hands and do what needed to be done. He was a plain, hard-working, high-thinking, right-minded, forward-looking, public-spirited citizen, who made himself felt in the community life".

Mr. Judson was elected a Member of the American Society of Civil Engineers on January 3, 1894.

JOHN CURTIS PATTERSON, M. Am. Soc. C. E.*

DIED JANUARY 6, 1927.

John Curtis Patterson was born in Germantown, Philadelphia, Pa., on July 12, 1857, the son of Joseph Patterson and Lavinia Horstman Patterson.

He received his early education at the Faires Classical Institute in Philadelphia and was the first student from this Institute to enter directly the Towne Scientific School of the University of Pennsylvania, as a member of the Class of 1878. He pursued his studies in Civil Engineering until December, 1877, in his Senior year, when he withdrew from college to take a position as Rodman with the Engineer Corps of the Madeira and Mamoré Railway, in Brazil, joining the ill-fated Collins Expedition sent out from Philadelphia to build this railway around the Falls of the Madeira River, a branch of the Amazon. Mr. Patterson was selected for this expedition because of his excellent physique and strong constitution, but he barely escaped the treacherous climate with his life, after suffering from the sickness which caused the abandonment of the enterprise for the time being.

On his return from South America in April, 1878, he entered the employ of the Philadelphia and Reading Railroad as Rodman and continued in this position until August, 1880, when he became Rodman on an Engineer Corps of the Pennsylvania Railroad.

In November, 1880, Mr. Patterson went to Colima, Mexico, where he served as Levelman and, later, as Division Engineer for the Mexican National Construction Company (the Manzanillo and Laredo Railway Company). Returning from Mexico in May, 1882, he was Assistant Engineer on surveys for the Gettysburg and Harrisburg Railroad, at Carlisle, Pa., until April, 1883, when he became Division Roadmaster of the Lebanon Division of the Philadelphia and Reading Railroad and continued in this position until February,

* Memoir prepared by Clark Dillenbeck, M. Am. Soc. C. E.

1885. He then became Secretary and Treasurer of the Philadelphia Drainage Construction Company, Limited, and, in November, 1887, he again returned to the Philadelphia and Reading Railroad Company as Assistant Engineer. He continued in this position until November, 1888, when he became Assistant Chief Engineer, and, later Chief Engineer, of the Poughkeepsie Bridge and Central New England and Western Railroad.

Following this Mr. Patterson was, successively, Engineer in Charge of Construction of the Quaker City Mortar Company's Plant in Philadelphia; Assistant Chief Engineer of Surveys for the Baltimore and Cumberland Railroad; Chief Engineer of the Pittsburgh and Eastern Railroad in Clearfield County, Pennsylvania; and Chief Engineer for the Government of Ecuador on the construction of the railway from Guayaquil to Quito, an interesting engineering achievement. After his return from Ecuador, until his death, Mr. Patterson was engaged in consulting work in Philadelphia.

He was married to Charlotte Dallas Morrell, by whom he is survived, with a sister, Mrs. Meredith Bailey, of Portland, Ore.

Mr. Patterson was a man of exceptionally genial and attractive personality, and a most delightful conversationalist, who will be greatly missed by his fellow engineers and his many friends. In his youth, he was a notable athlete, an excellent player of baseball and cricket, interested in the beginning of amateur baseball playing and also in that of the modern game of football.

He was a member of the Delta Psi Fraternity and of the St. Anthony's Clubs of Philadelphia and New York, and, at various times, of the Engineers' Club of Philadelphia, the Philadelphia Gun Club, the Germantown Cricket Club, the Huntington Valley Country Club, the Devon Golf Club, the Racquet Club, and the Philadelphia Country Club.

Mr. Patterson was elected a Member of the American Society of Civil Engineers on October 2, 1889.

HORACE AUGUSTUS SUMNER, M. Am. Soc. C. E.*

DIED DECEMBER 31, 1926.

Horace Augustus Sumner, the son of Robert S. and Mary Beals Sumner, was born at Stoughton, Mass., on March 18, 1845, and received his education in the public schools.

In July, 1864, he entered the Engineering Department of the Old Colony Railroad and was engaged as Chainman and Rodman on location and construction in Massachusetts and Vermont. He was also employed on similar work on the Portland and Ogdensburgh Railroad.

From April, 1868, to January, 1870, Mr. Sumner served as Transitman on location of the Burlington and Missouri River Railroad in Iowa. During this time he had charge of a division and constructed thirteen miles of the road near Corning, Iowa.

In April, 1870, he became Transitman on location of the Burlington and Missouri River Railroad, in Nebraska, from Crete to Fort Kearney, and

* Memoir prepared by R. S. Sumner, M. Am. Soc. C. E.

from July, 1870, to July, 1871, he was Engineer in charge of location and construction of the Burlington and Southwestern Railroad from Rulo to Humboldt, Nebr. From July, 1871, to November, 1873, he acted as Assistant Chief Engineer of the Burlington and Southwestern Railroad from Viele, Iowa, to Unionville, Mo., and from July, 1874, to May, 1875, he was Engineer of Public Works, Burlington, Iowa.

He then accepted the position of Division Engineer in charge of about ten miles of construction of the Knoxville Branch of the Chicago, Burlington, and Quincy Railroad, near Knoxville, Iowa, and carried on this work for a year.

Mr. Sumner was Chief Engineer of the Burlington and Southwestern Railroad, including the territory from Viele, Iowa, to Laclede, Mo., from May, 1876, to January, 1882, when he became Chief Engineer of the St. Louis, Keokuk, and Northwestern Railroad and the Chicago, Burlington, and Kansas City Railroad. He held this position, until May, 1884, with headquarters at Keokuk, Iowa.

From June, 1885, to March, 1886, he was Resident Engineer on construction of thirty miles of the Chicago, Burlington, and Northern Railroad, at East Dubuque, Ill. In June, 1886, he engaged on location work in Colorado, principally in the mountains, on what is now the Moffat Railroad over the Continental Divide, and in February, 1894, he accepted the position of Chief Engineer of the Florence and Cripple Creek Railroad. He held this position until April, 1895, when he was appointed State Engineer of Colorado, which office he held for two years.

Mr. Sumner served as Chief Engineer of the Alamogordo and Sacramento Mountain Railroad, in New Mexico, from November, 1897, to September, 1898, when he was appointed Chief Engineer of the El Paso and Northeastern Railway and was engaged on construction from El Paso, Tex., to Dawson, N. Mex. He resigned in 1902 to become Chief Engineer of the Moffat Railroad, in which position he made surveys from Denver, Colo, to Salt Lake City, Utah, and was also engaged in construction work from Denver to Steamboat Springs, Colo.

From 1912 to 1914, he was Chief Engineer on location surveys in California and Oregon for a company that was absorbed by the Southern Pacific Railway Company which built the line from Medford to Marshfield, Ore. From 1914 until his death he was engaged in general consulting work in Denver, Colo.

Mr. Sumner is survived by his widow and two sons, Robert S. Sumner, M. Am. Soc. C. E., and Lewis A. Sumner.

His past record is amply covered by the following tribute:

"Mr. Sumner * * * was an engineer of National prominence; but he was also a moral engineer, for he constructed his life on the divine lines of force, of integrity and uprightness, humility and gentleness, love and magnanimity, a reasonable and sincere faith in God. He was a towering pine of the forest. His many friends join in sympathy with his wife and sons, not only because they are our friends, but because we share with them their love of this great man who has died."

Mr. Sumner was elected a Member of the American Society of Civil Engineers on October 4, 1899, and served as Director of the Society from 1909 to 1911.

PETRUS WAHLMAN, M. Am. Soc. C. E.*

DIED MARCH 11, 1926.

Petrus Wahlman was born at Gefle, Sweden, on March 11, 1879. He received his early education at a school in Gefle and his technical training at the Royal University of Technology in Stockholm, Sweden, from which he was graduated in May, 1902, with the degree of Civil Engineer, having specialized in highway and hydraulic subjects. During his summer vacations he was engaged on railway location and survey work.

From June, 1902, to March, 1904, Mr. Wahlman held the position of Superintendent on the construction of the water-works and sewerage system for Hudiksvall, Sweden, the cost of which was $110 000, and from March to August, 1904, he conducted the work on the extension of the water-works of Oxelösund, Sweden.

In September, 1904, he investigated the stresses, through elongation tests, in two of the bridges of the State Railways of Sweden and in November, 1904, he became affiliated with the Hydraulic Engineering Company, Limited, of Stockholm, continuing in this position until May, 1907. During this period he made field studies, reports, and complete designs of water power plants in Sweden and Norway and supervised the design and construction of the water-works and sewerage system for Djursholm.

In 1907, Mr. Wahlman came to the United States and in June of that year secured a position as Draftsman with the firm of Vielé, Blackwell, and Buck, of New York, N. Y., on the design of the hydraulic plant of the Great Western Power Company.

From March, 1908, to February, 1909, he was engaged in work for the New York State Water Supply Commission, at Albany, making studies and designs for water power plants, storage dams, etc., and the utilization of the water powers of New York State for electrification. From March to June, 1909, he was occupied on hydraulic computations and the design for a power plant at Boulder, Colo., which was being erected by the Central Colorado Power Company. In June, 1909, he was engaged by the Ontario Power Company, of Niagara Falls, N. Y., to make hydraulic computations and designs for the extension of the Company's plant.

During the period from 1910 to 1913, Mr. Wahlman acted as Chief Engineer of the Construction Department of the Hydro-Electric Company of Tucuman, Argentine Republic, where he had responsible charge of the design and construction of an 18 000-h.p. water power plant on the Lules River and of two railways, the cost of which was $2 500 000. His duties included the building of a water-power plant for the utilization of two falls with a combined output of 40 000 h.p. situated at the foot of the Andes Mountains,

* Memoir compiled from information on file at the Headquarters of the Society.

approximately 600 miles from Buenos Aires, Argentine Republic. At times, he had as many as 1 000 workmen and many engineers in his employ.

In 1914, Mr. Wahlman made reports on engineering conditions in the South American Republics on the Pacific Coast and also visited the then more recently constructed water power plants in the United States, Canada, and Alaska. He was also interested in the development of the Speel River water power project in Alaska.

In 1915, he became associated with Mr. R. D. Johnson, and under the firm name of Johnson and Wahlman carried on consulting work in hydraulic engineering in New York. This firm also contributed toward the development of the water power projects on the Canadian side of Niagara Falls, where many new and valuable designs were carried out.

Late in December, 1924, he was forced to withdraw from his work and go South for his health, returning a little later to his home in Sweden. All his efforts to recover his strength were in vain, and he passed away on his birthday, March 11, 1926.

Through his engineering affiliations and his large circle of friends Mr. Wahlman quietly assisted many Swedish engineers who came to the United States in search of employment. Although he did not consider his services of great consequence, he will long be remembered by those men whose road to success was made easier by his advice.

He was very fond of sports, having participated as gymnast and swimmer in the first modern Olympic Games at Athens, Greece, in 1906. His favorite games were football and tennis and as a fancy skater he had won a championship.

His individuality was strongly marked by a most unusual self-control, evincing itself in a demeanor of faultless good breeding and gentility, kindliness and modesty of disposition, and unselfish consideration for others. His most outstanding characteristic in the opinion of all his acquaintances was that of a thorough gentleman in every sense of the word. He was a true son of the Nordic race and his many fine characteristics made him esteemed by all who knew him.

Mr. Wahlman was elected a Member of the American Society of Civil Engineers on December 6, 1915.

PHILIP ALBERT WELKER, M. Am. Soc. C. E.*

DIED DECEMBER 24, 1926.

Philip Albert Welker, the son of Philip and Maria (Pauly) Welker, was born at Toledo, Ohio, on June 1, 1857. He received his education in the public schools of Toledo, and was graduated from the High School in June, 1874. He then entered Cornell University from which he was graduated in June, 1878, with the degree of Civil Engineer.

After serving for a year as Assistant with the City Civil Engineer of Toledo, he was appointed in July, 1879, to a position with the United States

* Memoir prepared from information on file at Society Headquarters.

Coast and Geodetic Survey and was engaged on the transcontinental triangulation in Missouri.

From 1880 to 1890, Mr. Welker was occupied in primary triangulation and astronomical work in British Columbia, Washington, Southern California, Utah, Nevada, and Florida. He carried out work in precise levels in the States of Arkansas, Illinois, and Tennessee, as well as special surveys for the Navy Yard Site Commission in Escambia Bay, Florida, and for the location of the trans-meridianal line at Toledo. He also made hydrographic surveys of the harbor at Baltimore, Md., San Francisco Bay and tributaries, San Pablo Bay, and Suisun Bay, California, as well as of the Atchafalaya River, in Louisiana, Pensacola Bay and tributaries, in Florida, and, also, Maumee Bay, Ohio, for the State Fish Commission. During this period he made observations for the transit of Venus over the sun at Tepusquet, Calif., and a determination of the magnetic elements at the U. S. Coast and Geodetic Survey, as well as observations of the annular eclipse of the sun at San Francisco.

From 1891 to 1910, he carried out work in hydrographic surveying in Florida, Louisiana, and along the Atlantic Coast as far north as Maine. In 1893, he represented the Government of the United States in field observations connected with the survey of the boundary between Alaska and Canada. During the two years following, he continued to carry on boundary survey work in Alaska. In 1898, he was placed in command of the U. S. Coast and Geodetic Survey Steamer, *Bache,* continuing in this capacity until 1910, during which time he made surveys of Portsmouth Harbor and its approaches in New Hampshire and Maine, and along the coasts of the Atlantic Ocean and Gulf of Mexico, and in the Caribbean Sea.

In 1910, Captain Welker again represented the Government of the United States on a Commission of Engineers created for the purpose of determining the location and ownership of the Fort Myers Military Road, Virginia. While on a furlough of six months he organized the Welker Supply Company, at Cleveland, Ohio, and was elected President of the Corporation.

On the expiration of his furlough, Captain Welker was sent to Alaska where he determined the longitude of the Cities of Fairbanks and Tanana. On February 26, 1911, he was given charge of the Sub-Office of the U. S. Coast and Geodetic Survey at Manila, Philippine Islands, and until February 28, 1914, served as Director of Coast Surveys in the Islands. He also served as a member and Secretary of the Philippine Committee on Geographical Names and as a member of the Harbor Lines Commission of the Philippine Islands.

On October 10, 1914, he was appointed Assistant in charge of the office of the U. S. Coast and Geodetic Survey of Washington, D. C., and served in that capacity until July 1, 1917, when the designation was changed to Hydrographic and Geodetic Engineer in charge of the office. On July 1, 1920, he was designated as Personnel Officer of the U. S. Coast and Geodetic Survey and held that position until he was retired from active duty on August 1, 1921.

During approximately fifteen years of his service with the U. S. Coast and Geodetic Survey, Captain Welker was in command of vessels. His surveys of the coasts of Porto Rico and off the Isthmus of Panama formed the basis for some of the first complete navigating charts of these waters prepared by the Federal Government. His service included all branches of activity in the Bureau, from triangulation and astronomical observations in his earlier days, through the duties of Hydrographer, to responsible administrative positions in Manila and Washington.

He died at his home in Washington, D. C., on December 24, 1926, and is survived by his widow, formerly Gertrude M. Lanahan, of Baltimore, a son, Philip Lanahan Welker, and a daughter, Mrs. Howard R. Willis, of Cleveland, Ohio.

The conspicuous services of Captain Welker deserve more than the expression of regret for the loss of an able engineer. Through his systematic operations he played an important part in the security and extension of American commerce by determining and describing the dangers in its path. His results had always in evidence a distinctive mark of originality of execution scientifically accomplished, thereby increasing the efficiency and precision of the methods and work of the U. S. Coast and Geodetic Survey and the reputation which it has always sustained.

His kind and genial disposition gained for him the regard of all his associates in the field and office work of the Survey, and all those who dealt with him could rely implicitly on his innate sense of justice. In private relations, Captain Welker was high in the esteem of a large circle of acquaintances, and he will be remembered for the steady practice of all the virtues and amenities which adorn social life.

He was a member of the Washington Society of Engineers, Beta Theta Pi, Cosmos and Federal Clubs in Washington, the University Club of Manila, and the Philippine Club of New York, N. Y. He was also a member of the St. John's Protestant Episcopal Church, a Thirty-second Degree Mason, and a Knights Templar.

Captain Welker was elected a Member of the American Society of Civil Engineers on April 25, 1921.

OSCAR ERNEST BULKELEY, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 4, 1927.

Oscar Ernest Bulkeley was born on April 2, 1885, at Oneida, Ill. He was the son of John A. and Emma H. (Copley) Bulkeley. His father was a native of Canton, Ill., and his mother came from Pennsylvania. John A. Bulkeley was an artist and photographer until the failure of his health necessitated the removal of himself and family in 1890 to Grand Junction, Colo., where he engaged in ranch work. Mrs. Bulkeley died in 1897 and three years later the family moved to Riverside, Calif.

* Memoir prepared by Edward D. Rich, M. Am. Soc. C. E.

Oscar Ernest Bulkeley attended the public schools and High School at Riverside and was graduated in 1902. He then entered Knox College at Galesburg, Ill., and completed the course with first honors in three years, graduating in 1905 with the degree of Bachelor of Science.

His first engagement was with the Grand Junction Electric Light Company. With a desire for practical experience in mining he entered the employ of the Amalgamated Copper Company in 1906, at Butte, Mont. He began work in the copper mines as a laborer in order to become familiar with mining methods. His ability was soon recognized and at the end of four years he had been promoted to be General Purchasing Agent.

In February, 1910, Mr. Bulkeley entered the University of Illinois to further equip himself for engineering work for he then believed he wished to be a civil engineer. He was graduated in the Civil Engineering Course in 1912 and took the position of Engineer of the Water Department of Rockford, Ill. He was soon promoted to be Assistant Superintendent and remained on that work until January, 1916. His success at Rockford resulted in his selection as Superintendent of the Water Department at Jackson, Mich., where he continued until April, 1918. At that time he was sought by the Dupont Engineering Company on the construction of the mammoth Government powder plant near Nashville, Tenn. Soon afterward he was placed in charge of its water-works plant, where he remained until after the Armistice.

From February, 1919, to January, 1920, he was associated with the Ross P. Beckstrom Engineering Company, of Rockford, Ill. In January, 1920, Mr. Bulkeley went to Lansing, Mich., as Engineer for the Electric Light and Water Board. On May 1 of that year he was made Acting Superintendent and, on November 1, Superintendent of both the light and water activities of the City of Lansing. This position he held at the time of his death.

At the time Mr. Bulkeley went to Lansing the city was operating five water pumping stations. The Central Station was used both for pumping water and generating electricity. An old generating plant had been purchased from a private company and was not in efficient condition. This plant was changed almost entirely and converted into a municipal steam-heating station. The development of electricity was transferred to a fine new generating plant, realized largely through Mr. Bulkeley's wise judgment and energy. This installation cost about $2 500 000 and with the extensions now projected and under way the investment will be nearly $4 000 000.

In 1924, a new pumping station, wells, and storage reservoir which furnishes a large part of the municipal water supply was completed under Mr. Bulkeley's direction, at a cost of nearly $350 000.

The Electric Light and Water Board placed great confidence in Mr. Bulkeley's advice in extending and improving the service. He was untiring in his efforts to work out a new street lighting system, in the removal of overhead wires in the business district, and in the development of an adequate water supply, all with broad vision of the future needs of the city.

He was always unselfishly interested and active in the upbuilding of his city and gave generously of his time to those civic, social, professional, and religious activities which are beneficial to mankind. He and Mrs. Bulkeley

were members of and active in the work of the Plymouth Congregational Church. His unfailing courtesy and kindliness endeared him to a host of friends who mourn his departure as a keen personal loss.

Mr. Bulkeley was taken ill with influenza on December 14, 1926. Pneumonia developed and caused his death on January 4, 1927.

On December 21, 1912, he was united in marriage to Edna S. Best, at Chicago, Ill., as a culmination of a romance of their school days in California. His widow and two children, Mary Louise and William Warren, survive him.

He was a member of the Phi Gamma Delta and Tau Beta Pi Fraternities; Lansing Lodge No. 33 F. and A. M.; Lansing Rotary Club; American Water Works Association; and the Lansing Engineers' Club, of which he was President in 1924.

Mr. Bulkeley was elected an Associate Member of the American Society of Civil Engineers on October 9, 1917.

HENRY GARNER HARPER, Assoc. M. Am. Soc. C. E.*

DIED MARCH 24, 1927.

Henry Garner Harper was born at Ashland, Pa., on November 15, 1891, the son of Henry C. and Catherine Harper. He received his public school education in Ashland and was graduated from the High School in 1908.

From 1908 to 1911, Mr. Harper was employed as Chainman at Ashland by the Philadelphia and Reading Coal and Iron Company. In the autumn of 1911 he entered Pennsylvania State College, from which he was graduated in 1915 with the degree of Bachelor of Science in Civil Engineering, as well as Bachelor of Science in Highway Engineering.

After his graduation from college he entered the service of the Pennsylvania Department of Highways, with which he was continuously employed until the time of his death, except during the period of his military service in the World War.

Mr. Harper's first assignment in the State service was for one year as Draftsman at the Harrisburg Office. This was followed by two years as Assistant Engineer in the Bureau of Township Highways. In June, 1918, he entered the Government military service and was commissioned a First Lieutenant in the Artillery. He served until he was honorably discharged early in 1919.

In May, 1919, Mr. Harper resumed work with the Pennsylvania Department of Highways as Resident Engineer in charge of construction work in McKean County. On January 1, 1920, he was appointed as District Engineer with headquarters at Sunbury. In August, 1921, he was transferred to Bedford, where he filled assignments as District Engineer and Assistant District Engineer, respectively, until June, 1923, when he was transferred to Hollidaysburg as District Engineer in charge of highway work in Blair, Cambria, and Huntingdon Counties. The latter assignment he was filling at the time of his death.

* Memoir prepared by S. W. Jackson, M. Am. Soc. C. E.

As District Engineer, Mr. Harper had charge of both construction and maintenance work, as well as of all other departmental activities. The Pennsylvania Department of Highways carried out its largest construction program during the period that Mr. Harper served as District Engineer, and many large construction projects were completed under his direction. In the midst of these activities he was stricken with spinal meningitis, following a mastoid operation, and died at the Altoona Hospital on March 24, 1927.

In 1917, Mr. Harper was united in marriage with Pauline Sharpless who, with a son and daughter, survives him.

He had a charming personality, with a keen sense of humor and a ready smile that won him many friends. He was known among his associates as a man of sterling integrity and possessed of the highest ideals of professional ethics.

After his graduation he kept in close touch with his Alma Mater and was particularly interested in the welfare of the Sigma Pi Fraternity of which he was a member. He was also a member of the American Legion, Fort Fetter Post No. 516, Hollidaysburg, Washington Lodge No. 265, F. and A. M., the Benevolent and Protective Order of Elks, of Bloomsburg, Pa., and the Kiwanis Club, of Hollidaysburg.

Mr. Harper was elected an Associate Member of the American Society of Civil Engineers on October 11, 1920.

HOWARD B. MERRICK, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 14, 1926.

Howard B. Merrick was born at Wrightstown, Pa., on January 2, 1871, the son of George T. and Mary Ann (Short) Merrick. His father was a Pennsylvania Quaker and his mother was of Vermont stock, but not a Quaker. As a youth he attended the Friends' School at Wrightstown and also, for a time, the Newtown High School, but did not complete the High School course.

In the fall of 1893, he entered the University of Michigan by examination, but not having been prepared in all subjects necessary for entrance without conditions, he spent part of the time of the first semester in the Ann Arbor High School in order to complete his entrance requirements. He depended largely on his own earnings to pay the expenses of his University course, and each of the summers of 1894 to 1897, inclusive, he spent in surveying work, serving as Rodman for the Elkhart and Western Railway Company the first summer and for the Great Northern Railway Company the other three summers.

Mr. Merrick was graduated from the College of Engineering of the University of Michigan in 1898, having taken the degree of Bachelor of Science in Civil Engineering. One of the years intervening after his entrance into the University was spent in the College of Literature, Science, and the Arts.

* Memoir prepared by James B. Pollock, Professor of Botany, Univ. of Michigan, Ann Arbor, Mich.

The summer following his graduation he was Masonry Inspector for the Great Northern Railway Company, and from October, 1898, to November, 1902, he served successively as Levelman, Transitman, Assistant Engineer, and Engineer in Charge of Location and Construction for the same Company, in Minnesota, North Dakota, and Montana. In the former State his work took him into the iron country on the Mesaba Range where he became interested in iron mining as carried on in that district, and from December, 1902, to September, 1903, he was Engineer in Charge of Drills for E. J. Longyear and Company. His work in this connection was the exploration of the territory in the vicinity of Hibbing, Minn., for the purpose of locating bodies of iron ore.

Five years after his graduation Mr. Merrick was called by his Alma Mater to serve as Instructor in Surveying, and began his teaching in the fall of 1903. He was advanced to Assistant Professor of Surveying in 1906 and to Associate Professor in 1918. In 1913, he was granted the degree of Civil Engineer, presenting as part of the requirement for that degree a thesis on "The Welfare of Railways in the United States Since the Passage of the Interstate Commerce Act". From 1922 until his death, he was Associate Professor of Geodesy and Surveying.

During his term of service at the University, Professor Merrick was engaged in some outside practice in general surveying, and undertook several special pieces of work. In June, 1913, he was with Mr. M. W. Thompson preparing data for use in the Duluth, South Shore and Atlantic Railway rate case, and in August and September of the same year he worked on the valuation of the Lake Shore and Michigan Southern Railway for the same firm. In the employ of H. E. Riggs, M. Am. Soc. C. E., he was in charge of the computation of earthwork on the appraisal of the Pere Marquette Railway, from June to October, 1914. He also worked at appraising real estate belonging to the Eastern Michigan Edison Company, from September, 1915, to February, 1916.

Besides his teaching Professor Merrick served the College of Engineering in several other capacities. He was in charge of the Summer Camp of the Junior Civil Engineering students from 1904 to 1911, and was the first Head Mentor in the Freshmen Mentor System which was inaugurated at the Engineering College in 1913. He also had charge of the Freshmen Assemblies in the College of Engineering in 1917 and 1918. Shortly before his death, he had completed a committee assignment connected with the current study of engineering education. His professional services were not only utilized by the University of Michigan and by various railway corporations, but a foreign nation, China, availed itself of his engineering talents.

In 1918, Professor Merrick was one of three men chosen from the Department of Geodesy and Surveying of the College of Engineering to go to China under the direction of Joseph Ripley, M. Am. Soc. C. E., an eminent graduate of the same College, in the employ of the American International Corporation, to aid the Chinese Government in the improvement of the Grand Canal. From August, 1918, to November, 1919, Professor Merrick served as one of the Principal Assistant Engineers to the Grand Canal Improvement Board in China. He was also in charge of precise level work, covering a

distance of 325 miles, and of the office work in the reduction and plotting of the field notes of the topographic survey parties.

At the request of the Chinese Government he was transferred in December, 1919, to the Commission for the Improvement of the River Systems of Chihli, a large province of Northeast China. He served as Engineer in Charge of Surveys, and, as such, re-organized the field and office forces and had complete supervision of the surveying and mapping of 30 000 sq. miles in the Province of Chihli. He directed a force of about 400 men, of whom 90 were engineers. A majority of the chiefs of parties were engineers who had been educated in America, England, or Japan, while many of the subordinates were graduates of Chinese engineering colleges having American teachers. Professor Merrick served the Commission for the Improvement of the River Systems of Chihli until June, 1922, having spent almost four years in China. He was then obliged to return to his duties at the University of Michigan.

That his work was done in a manner eminently satisfactory to all concerned, is attested by several facts. The Chinese Government was anxious to retain his services. Mr. Ripley, under whose direction Professor Merrick went to China, has expressed high appreciation of his accomplishments. Each of two groups of engineers under his charge in China, the staff and the substaff, presented him with a silver loving cup, engraved with an expression of "their high esteem and affection for him as their Chief during three years of association in the Survey Department of the Chihli River Commission." After his departure for the United States the Chinese Government awarded him a Fifth Class Chiao Hu Decoration, in recognition of his valuable services. Mr. G. G. Stroebe, a graduate of the University of Michigan, who became Chief Survey Engineer of the Yangtse River Commission about the time Professor Merrick returned to the United States, writes that "Professor Merrick's surveys are considered equal to the very best of those executed by the U. S. Coast and Geodetic Survey in this country." Mr. Stroebe also remarks, "They all say here that Mr. Merrick was a past master in dealing with delicate situations in China."

In his family life he was an ideal husband and father. He was married on December 12, 1900, to M. Grace Green, of Holly, Mich. Mrs. Merrick survives him, as do two daughters, Ruth Carolyn, a Sophomore in the College of Architecture, and Alice Mary, a Senior in the University High School. Two sisters are left also to mourn his loss.

A man's personality is perhaps best appreciated by his neighbors and intimate friends, a teacher's by his students and colleagues. Judged by them, Professor Merrick ranks very high. Dean Mortimer E. Cooley, M. Am. Soc. C. E., of the Colleges of Engineering and Architecture, said* of him:

"Since the very beginning he was closely in touch with our students and enjoyed their confidence. His equable temperament, his kindly heart and his charming manners will long be remembered. He will be a great loss to the University, particularly to the Geodesy and Surveying Department, and his place will be hard to fill."

Neighbors wrote of him, "He was one of the finest, best, kindest men I ever knew." "His consideration for others was such an outstanding charac-

* *Michigan Daily*, December 15, 1926.

teristic." A classmate reports, "Howard was one of the most esteemed members of our class of '98 Engineers." A student of a later time says, "I gained Professor Merrick's friendship while a student in his classes, and have valued it above most of my other friends."

Although reared a Quaker Professor Merrick was not so narrow in his religious views but that he could ally himself with other religious organizations and bear his share of their responsibilities, in a community where the Quakers had no organization. In recent years he was leader of a troop of Boy Scouts and freely gave them of his time, his wisdom, and his friendship.

In his professional relations he was a member of the Association of Chinese and American Engineers, the Association of American Engineers, and the Michigan Engineering Society.

Professor Merrick was a member of the Phi Sigma Kappa Fraternity, and of the Tau Beta Pi, an honorary engineering society, the University Club, and Huron Hills Golf Club.

He was also a member of the Masonic Order, and helped to found a lodge in Tientsin, to which he transferred his membership while he was in China, and which he served in various capacities. After returning to Ann Arbor, he joined with others in the establishment of a third local lodge and at the time of his death was holding an important office therein.

In a summary of Professor Merrick's life it may be said in all truth and without exaggeration, that he was an ideal husband and father; the finest, truest, and most lovable friend; a man ever thoughtful and considerate of others; an excellent citizen, an inspiring teacher, an efficient executive; a painstaking and exact worker in his profession, a civil engineer of high standing; a courteous advocate, pressing his point with reason and kindness rather than with aggressive domination; gentle in manner, but adamant in his advocacy of what he conceived to be right, like his ancestors the Friends or Quakers, accepting the "inner light" as his ultimate guide and authority.

Professor Merrick was elected an Associate Member of the American Society of Civil Engineers on July 1, 1909.

ROBERT BRUCE ROBINSON, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 1, 1927.

Robert Bruce Robinson was born at Mount Holly Springs, Pa., on March 3, 1878. He received his technical education at the University of Illinois, and entered the service of the Union Pacific Railroad Company in April, 1899, as a Rodman on the Kansas Division. In 1904 he went to the Oregon Short Line Unit of the Union Pacific System where he was engaged for several years in locating and constructing new branch lines, one of which was the North Side Branch, extending from Rupert to Bliss, Idaho.

* Memoir prepared by E. W. Plotz, Chf. Clerk, Maintenance of Way Dept., Union Pacific R. R., Omaha, Nebr.

After some time spent as a Special Engineer on investigation work, he was made Division Engineer of the Idaho Division of the Oregon Short Line, and, in 1916, accepted the position of Engineer, Maintenance of Way, for that railroad, with headquarters at Pocatello, Idaho. He served in this capacity until May, 1919, when he was promoted to be Engineer, Maintenance of Way, of the Union Pacific Railroad with headquarters at Omaha, Nebr., which position he held at the time of his death.

He died at St. Joseph's Hospital, Omaha, on February 1, 1927, from blood-poisoning resulting from the extraction of an infected abscessed tooth.

Mr. Robinson was an indefatigable worker, who gave unstintingly of his time and energy because of an intense love for the upbuilding of the great railroad system which it had been his part, as an engineer and builder, to introduce to many unsettled sections in the mountain States of the West, where prosperous communities have since sprung into existence.

He possessed that priceless executive trait of being able to get the best from his men at all times. In the strenuous work in which he was engaged, emergencies are the natural order, and he himself was always on the scene at the earliest possible moment to direct operations whenever serious floods and other troubles threatened to endanger the track. His personality was such, however, that even under the most trying conditions, when some one had failed, his kindly criticism produced more lasting results than all the harsh words that could ever be spoken, and no employee ever left a conference with Mr. Robinson with a feeling of bitterness or resentment.

It can be said with all sincerity that no general officer ever had a firmer grip on the affections of his men, and he was likewise warmly respected by the executive officers of the Company.

His widespread popularity was attested by the large attendance of friends, including many from far distant points, in addition to those of the Union Pacific family, at the funeral services held February 3d, at the Scottish Rite Cathedral, in Omaha, as well as by the exceptional number of beautiful floral offerings. On account of his love for the mountains, his remains were taken to Salt Lake City, Utah, and laid at rest on February 7, 1927, in the shadow of the stately Wahsatch Mountain Range. The services at Salt Lake City were the occasion for a large outpouring of friends in that community where much of his earlier work had centered.

Mr. Robinson was a member of the American Railroad Association, the American Railway Engineering Association, the American Railway Bridge and Building Association, and other technical societies. He was also a member of various Masonic bodies and of El Korah Temple Shrine, at Boise, Idaho.

He is survived by his widow, Mrs. Audley B. Robinson, and two children, Eleanor and Robert. His parents, Dr. and Mrs. R. E. Robinson, two sisters, Laura Robinson and Mrs. P. H. Simpson, all of Morrison, Ill., and two brothers, Arthur Robinson, of Pocatello, and Edward Robinson, of Cleveland, Ohio, also survive him.

Mr. Robinson was elected an Associate Member of the American Society of Civil Engineers on July 9, 1912.

CLARENCE HARD THOMPSON, Assoc. M. Am. Soc. C. E.*

DIED MARCH 10, 1927

Clarence Hard Thompson, the son of Lucella and Guy Thompson, was born at Odessa, N. Y., on December 2, 1867. After completing his studies in the public schools, he entered the Classical Course of the Cortland, N. Y., Normal High School from which he was graduated in January, 1888.

After having taught school for three years, Mr. Thompson, in 1891, entered the Engineering Department of the Groton Bridge and Manufacturing Company, of Groton, N. Y. As a result of his marked ability, he was made Chief Draftsman in 1894, which position he held until 1901 when he accepted a similar engagement with the Owego Bridge Company, of Owego, N. Y.

In 1903, Mr. Thompson entered the employ of the Solvay Process Company, of Syracuse, N. Y., as a Structural Engineer and Computer, where he remained until ill-health forced him to retire in February, 1926. During this period he was engaged in the design and supervision of plans for many buildings of structural steel framework, brick, and reinforced concrete.

He was very methodical in his business life and left a complete record of computations covering a period of twenty-three years. His keen judgment augmented by constant study, enabled him to solve many complicated problems in his line of work. Among his associates he was considered an authority on engineering projects dealing with construction work.

In disposition and manner Mr. Thompson was modest and cheerful, but firm in facing unpleasant situations, stoutly maintaining his opinion when he knew he was in the right.

He was a faithful member of the Good Will Congregational Church, of Syracuse, where he taught a large Bible Class for more than twenty years. He was married in 1896 to Mary H. Chapman, who survives him.

Mr. Thompson was elected an Associate Member of the American Society of Civil Engineers on September 7, 1904.

* Memoir prepared by Charles E. Harris, Esq., Syracuse, N. Y.